University of Kufa

Faculty of Engineering

Structures and Water Resources Engineering

Hydraulic Structures Lectures – Fourth Class

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References:

1. Theory and Design of Irrigation Structures – Volume I by Gupta. 6th Edition. 1993.

2. Hydraulic Structures by Novak. 4th Edition 2007.

3. Irrigation, Water Power and Water Resources Engineering by K.R. Arora. 4th Edition 2007.

Syllabus:

First Semester:

Chapter One: Introduction

Chapter Two: Theories of Creep

Chapter Three: Hydraulic Jump and Energy Dissipation Structures

Chapter Four: Hydraulic Design of Regulators

Chapter Five: Design of pipes and Box Culverts

Second Semester:

Chapter Six: Design of Inverted Siphon

Chapter Seven: Weirs

Chapter Eight: Design of Gates

Chapter Nine: Design of Dams

Chapter Ten: Design of Spillway

Chapter One

Introduction

1.1. Introduction

A hydraulic structure is a structure submerged or partially submerged in any body of water, which disrupts the natural flow of water. They can be used to divert, disrupt or completely stop the flow. An example of a hydraulic structure would be a dam, which slows the normal flow rate of the river in order to power turbines. A hydraulic structure can be built in rivers, a sea, or any body of water where there is a need for a change in the natural flow of water.

Hydraulic structures may also be used to measure the flow of water. When used to measure the flow of water, hydraulic structures are defined as a class of specially shaped, static devices over or through which water is directed in such a way that under free-flow conditions at a specified location (point of measurement) a known level to flow relationship exists. Hydraulic structures of this type can generally be divided into two categories: flumes and weirs

1.2. Types of Hydraulic Structures

According to the purpose of its function

1- **Storage Structures:** the function is to store water such as Dams & Tanks.
2- **Conveyance Structures:** the function is to convey water from place to another such as pipelines, Siphons, Culverts, Tunnels, aqueducts and open channels.
3- **Flow Diversion Structures:** the function is to regulate and divert the quantities of flow to another structures or canals such as barrages and regulators.

4- **Flow measurement structures:** the function is to measure the flow passing through it such as Weirs, Orifices, nozzles, Venturi and Parshall flume.
5- **Energy dissipation structures:** the function is to protect the floor of hydraulic structure from erosion and damage due to severe waves which impact with the body of structure such as Stilling basins, Surge tanks, Check dams and vertical drop.
6- **Power Stations:** the function of these structure is to convert energy from a case to another such as Pumps, Turbines & Rams.
7- **Sediment and Chemical Control Structures:** the function is to control or remove sediments and other pollutants such as Sedimentation tanks, Screens, Traps, Filters & mixing basins.
8- **River Training and Waterway Stabilization Structures:** the function is to maintain river channel and water transportation such as Levees, Cutoffs, Locks, Piers Dikes, Groins, Jetties and Revetments

Remark: For any hydraulic structure to design, we must study the following:

1- Hydrologic studies.
2- Hydraulic studies.
3- Structural studies.

1.3. Steps for Design of Hydraulic Structures

To construct any hydraulic structure, the following steps must be considered:

1. Prepare information for design.

a. The precise function of design.
b. Discharge (Max. & Min.) Use 1.2 Q max. discharge & 0.7 Q for min. discharge.
c. Head loss.
d. U/S & D/s canal.

2. Determine the best location of the structure.
3. The shape of approach and the other components of the structure.
4. The requirements of water-way.
5. Protection against scouring.
6. The best method of dissipation energy.
7. Forces acting on various parts of the structure, Hydraulic forces (hydrostatic pressure, dynamic forces) & other forces, live loads, dead loads, earth pressure.

1.4. Site Conditions

In design of any structure, site condition have be taken into accounts:-

1. Soil properties.
2. Ground water.
3. Soil strength parameter.
4. Permissible bearing pressure.
5. Permeability.
6. Mineral content (especially sulphates) to both soil & ground water.

1.5. Structures on Gypsum Soils

Regardless of the mode occurrence, the effect of saturation of the pore space with relatively fresh water is that gypsum as taken into solution. Permeability is increased with consequent increase in seepage rate, soil strength is reduced, cavities are formed in the soil structure and foundation failure by piping or undermining may occur.

Where site investigation shows significant gypsum concentration in the underlying soil strata, every efforts should be made to relocate structures to more favorable locations. Channels U/S & D/S of the structure should be lined and particular attention paid to joints to ensure that water tights is maintained.

1.6. Percolation beneath Heading up of Hydraulic Structures

The hydraulic Structures such as barrages, regulators, culverts, etc…, may either founded on an impervious solid rock foundation or a pervious foundation. It is subjected to seepage of water beneath the structure in addition to all other forces to which will be subjected. When founded on un impervious rock foundation, the water seeping below the body of the hydraulic Structure, endangers the stability of the structures may cause its failure.

1.7. Causes of Failure of Hydraulic Structures Founded on Pervious Foundations

1.7.1. Failure by Piping or Undermining

Water starts seeping under the base of hyd. Structure. It starts from U/S side and tries to exit at the D/S end of the impervious floor. At the point of the exit, the exit gradient may become more than the critical gradient, in which cause, the water starts dislodging the soil particles & carrying it away with it causing formulation a hole in the subsoil. So, formed resulting in the failure of the structure.

Piping can have prevented by the following methods:

a. By providing sufficiently long impervious floor

This long length will reduce the exit velocity & exit gradient. As the water has to travel along distance beneath the floor, its head will sufficiently have lost before it exits & its velocity will be such that it has cannot wash away any soil or sand particles.

b. By providing piles at both U/S and D/S ends:

This measure also results is increasing the path of the travel of seepage water & hence it decreases its exit velocity & exit gradient.

1.7.2. Failure by Direct Uplift

The water seeping below the structure exerts on uplift pressure on the floor of the structure if this pressure is not counter balance by the weight of concrete or masonry floor. The structure will fail by a rupture of a part of the floor. The pervious concept of the hydraulic structure due to subsurface flow where introduce by many engineers on the bases of experiments & the research work.

Chapter Two

Theories of Creep

2.1. Bligh's Creep Theory

It is directly depended on the possibilities of percolation in porous soil on which the floor (apron) is built. Water from upstream percolates and creeps (or travel) slowly through weir base and the subsoil below it. The head lost by the creeping water is proportional to the distance it travels (***creep length***) along the base of the weir profile. The creep length must be made as big as possible so as to prevent the piping action. This can be achieved by providing deep vertical cut-offs or sheet piles.

According to Bligh's theory, the total creep length, L = b + 2(d_1 + d_2 + d_3) (Figure (2.1)), where b = L_1+L_2. If H is the total loss of head, then the loss of head per unit length of the creep shall be:

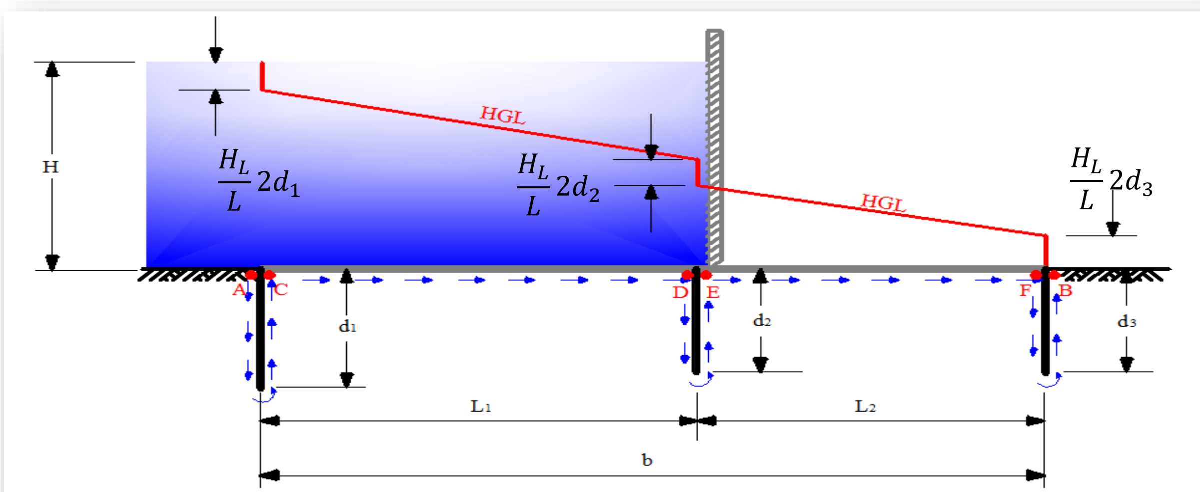


Figure (2.1): The Path of Creep Line (Bligh's Theory)

$$\mathbf{C = \frac{H}{L} = \frac{H}{b + 2(d_1 + d_2 + d_3)}} \quad \text{..... (2-1)}$$

$$\mathbf{H = W.L_{(u/s)} - W.L_{(d/s)}} \quad \text{..... (2-2)}$$

Bligh called the loss of head per unit length of creep as *Percolation coefficient* or *hydraulic gradient (C)*. The reciprocal, (L/H) of the percolation coefficient is known as the *coefficient of creep(C')*.

2.1.1. Assumptions

1. Hydraulic slope or gradient is constant throughout the impervious length of the apron.
2. The percolating water creep along the contact of the base profile of the apron with the sub soil losing head enroute, proportional to length of its travel. The length is called ***creep length***. It is the sum of horizontal and vertical creep.
3. Stoppage of percolation by cut off (pile) possible only if it extends up to impermeable soil strata.

For more explaining (See figure 2.2):

Total creep length (L) = $2d_1 + L_1 + 2d_2 + L_2 + 2d_3 = (L_1 + L_2) + 2\,(d_1 + d_2 + d_3)$

Head loss per unit length (hydraulic gradient)

$$C = \frac{H}{L} = \frac{H}{(L_1 + L_2) + 2\,(d_1 + d_2 + d_3)}$$

Head loss occurs on upstream cutoff $= \frac{H_L}{L} \times 2d_1$

Head loss occurs on intermediate cutoff $= \frac{H_L}{L} \times 2d_2$

Head loss occurs on downstream cutoff $= \frac{H_L}{L} \times 2d_3$

Head at point C (H_C)

$$H_C = \frac{H}{L}\,(L - 2d_1) = H - \frac{2d_1}{L}$$

Hydraulic gradient drop at upstream cutoff = $H - H_C = H - \left(H - \frac{2d_1}{L}\,H\right) = \frac{H}{L}\,2d_1$

Head at point E (H_E)

$$H_E = \frac{H}{L}(L_2 + 2d_3) = \frac{H}{L}(L_2) + \frac{H}{L}(2d_3)$$

Hydraulic gradient drop at intermediate cutoff = H – H_E = $H - \left(\frac{H}{L}(L_2) + \frac{H}{L}(2d_3)\right)$

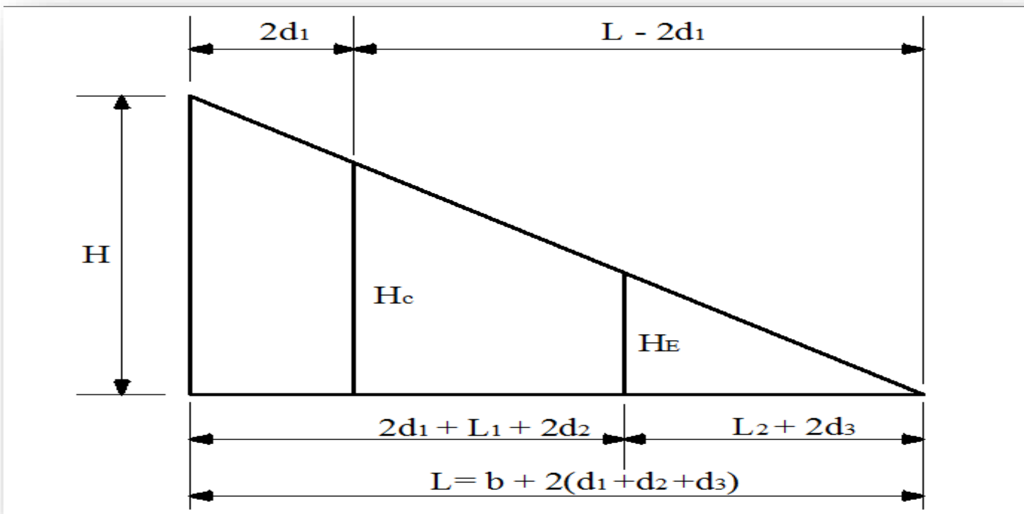


Figure (2.2): Head Calculations at Multi Key Points

2.1.2. Safety against Piping or Undermining

Safety against piping can be ensured by providing sufficient creep length given by:

$$\mathbf{L = C\,H} \qquad \text{….. (2-3)}$$

C: Bligh's coefficient for the soil.

$$\frac{H}{L} = \frac{1}{C}$$

Table (2.1): Recommended Safe Hydraulic Gradients

No.	Type of soil	Value of C	Safe exit gradient less than
1	Light sand and mud	18	$\frac{1}{18}$
2	Fine sand, alluvial soil	15	$\frac{1}{15}$
3	Coarse grained sand	12	$\frac{1}{12}$
4	Sand mixed with boulders and gravel	9-5	$\frac{1}{9}$ to $\frac{1}{5}$

2.1.3. Safety against Uplift Pressure

The ordinate of hydraulic grade line above the bottom of the floor represent the residual uplift water head at each point. Say for example; if at any point the ordinate of H.G.L above the bottom of the floor is ***1m***, then ***1m*** head of water will acts as uplift at this point. If the uplift head at any point is (H_p), then water pressure equal to (H_p) meters will acts at this point and has to be counter balanced by the weight of the floor of thickness say (t_p).

Uplift pressure = $\gamma_w\ H_p$

Where γ_w is the unit weight of water ($\gamma_w = \rho\ g$)

Downward pressure = $(\gamma_w\ G)\ t_p - \gamma_w\ t_p$

Where G is the specific gravity of the floor material.

G is approximately taken as 2.24 to 2.4

For equilibrium condition:

$$\gamma_w\ H_p = (\gamma_w\ G)\ t_p - \gamma_w\ t_p$$

$$H_p = G\ t_p - t_p = t_p\ (G - 1)$$

$$\mathbf{t_p = \frac{H_p}{G-1}} \qquad \text{..... (2-4)}$$

Example (2.1): Find the hydraulic gradient and uplift pressure and the thickness of floor at a point 15 m from the upstream end of the floor in the figure (2.3).

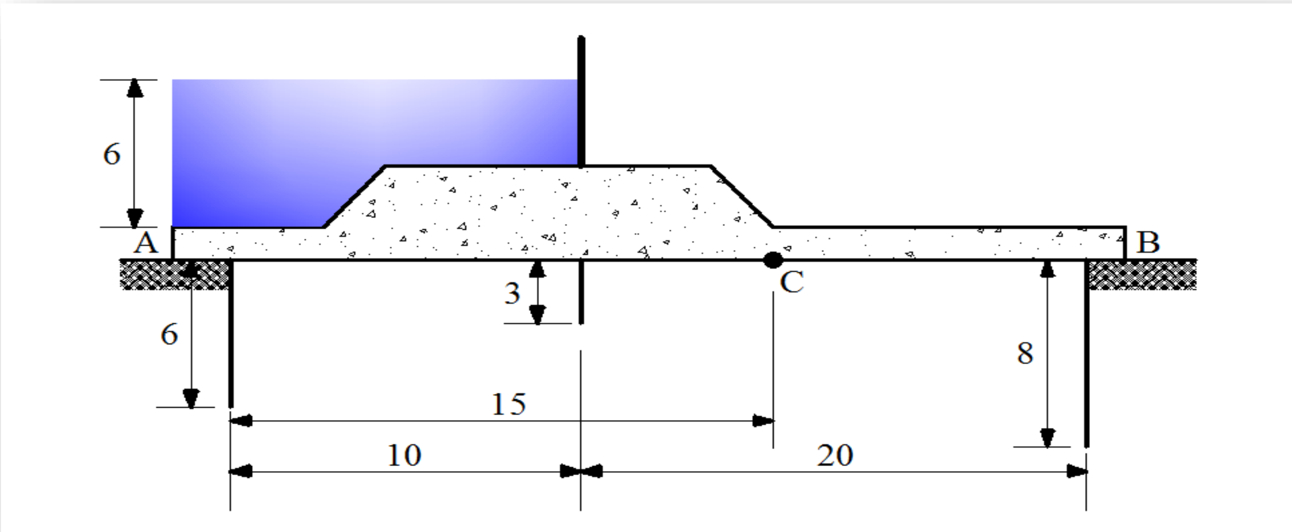


Figure (2.3): Definition Sketch for Example (2.1)

Solution

Water percolates at point *A* and emerges at point *B*

Total creep length (L) = 2×6+10+ 2×3+ 20+ 2×8 = 64*m*

Head of water on structure (H) = 6 *m*

Hydraulic gradient = H/L = 6 /64 = 1/C = 1/10.67

According to Bligh's theory, the structure would be safe on sand mixed with boulders & Gravel

Creep length up to point $C = L_1$ = 2×6+ 2×3+15 = 33*m*

The residual uplift pressure at the point C under consideration

$$H_C = \frac{6}{64}(64 - 33) = 2.91 \text{ m}$$

The thickness of floor at C

$$t_c = \frac{H_C}{G_c - 1} = \frac{2.91}{2.4 - 1} = 2.08 \text{ m of concrete}$$

Homework No. 1: For the hydraulic structure shown below:

1- Sketch the H.G.L from U/S to D/S 2. Find the uplift pressure at key points ④&⑦.

2- Find the thickness of floor at key point ⑥.

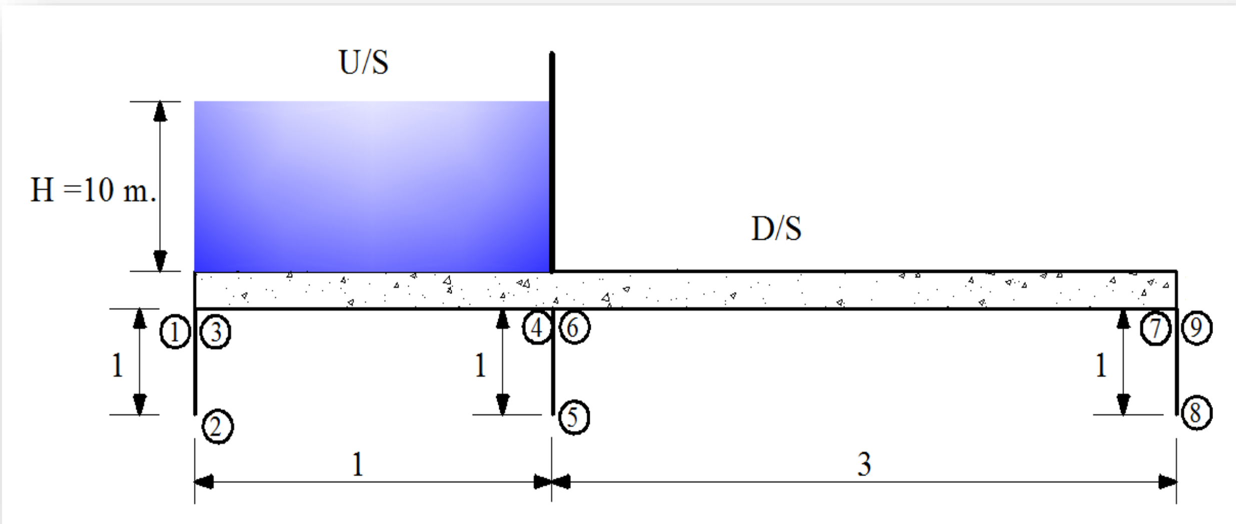


Figure (2.4): Definition Sketch for Homework # 1

2.2. Lane's Weighted Creep Theory

Bligh, in his theory, had calculated the length of the creep, by simply adding the horizontal creep length and the vertical creep length, thereby making no distinction between the two creeps. However, Lane, on the basis of his analysis carried out on about 200 dams all over the world, stipulated that the horizontal creep is less effective in reducing uplift (or in causing loss of head) than the vertical creep. He, therefore, suggested a weightage factor of 1/3 for the horizontal creep, as against 1.0 for the vertical creep.

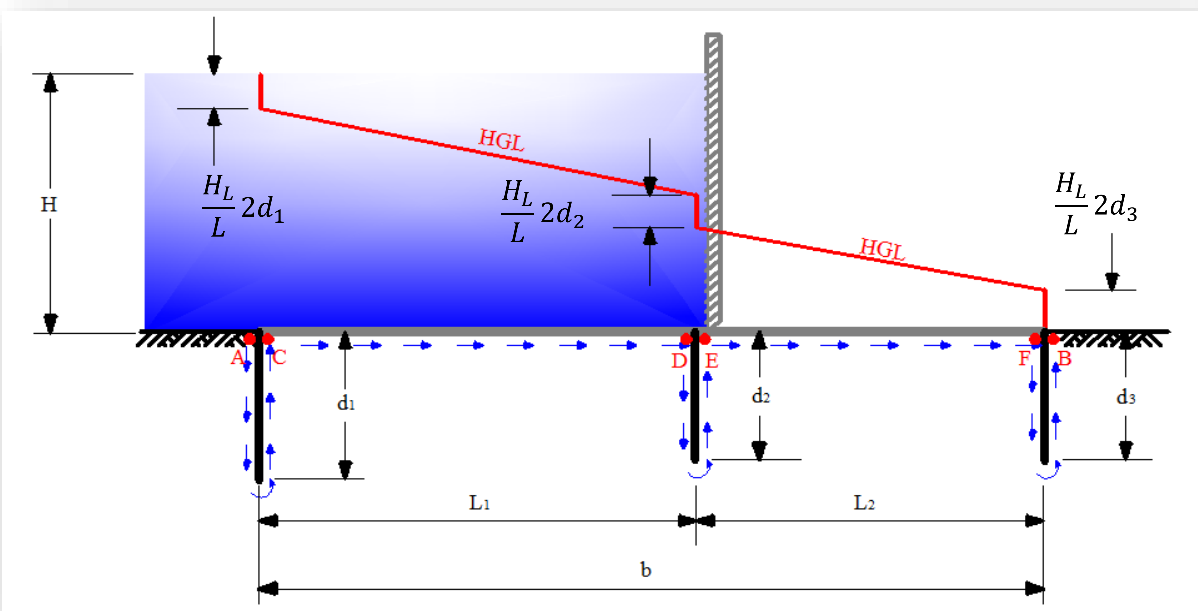


Figure (2.5): The Path of Creep Line (Lane's Theory)

According to figure (2.5) above, the total Lane's creep length (L_l) is given by:

$L_1 = (d_1 + d_1) + (1/3)\ L_1 + (d_2 + d_2) + (1/3)\ L_2 + (d_3 + d_3)$

$= (1/3)\ (L_1 + L_2) + 2(d_1 + d_2 + d_3) = (1/3)\ b + 2(d_1 + d_2 + d_3)$

To ensure safety against piping, according to this theory, the creep length Ll must not be less than C_1H_L.

Where H_L is the head causing flow, and C_1 is Lane's creep coefficient given in table 2.

To ensure safety against piping

$L_w > C_1 H$

Hydraulic gradient $\frac{H}{L_w}$ should be less than $\frac{1}{C_1}$

$$\frac{H}{L_w} < \frac{1}{C_l}$$

Table (2.2): Values of Lane's Safe Hydraulic Gradient for different types of Soils

No.	Type of soil	C_1	Safe exit gradient less than
1	Very fine sand or silt	8.5	$\frac{1}{8.5}$
2	Fine sand	7.0	$\frac{1}{7.0}$
3	Coarse sand	5.0	$\frac{1}{5.0}$
4	Gravel & sand	3.5 to 3.0	$\frac{1}{3.5}$ to $\frac{1}{3.0}$
5	Boulders, gravel and sand	3.0 to 2.5	$\frac{1}{3.0}$ to $\frac{1}{2.5}$
6	Clayey soil	3.0 to 1.6	$\frac{1}{3.0}$ to $\frac{1}{1.6}$

Example (2.2): You are working as a consultant for an engineering company, and you have received a design of a barrage structure on a river shown in figure below. It is required to check if the thickness at points X, Y and Z is sufficient to counteract the uplift pressure (G = 2.4), and check safety against piping if the soil type is coarse sand (C_1 =5).

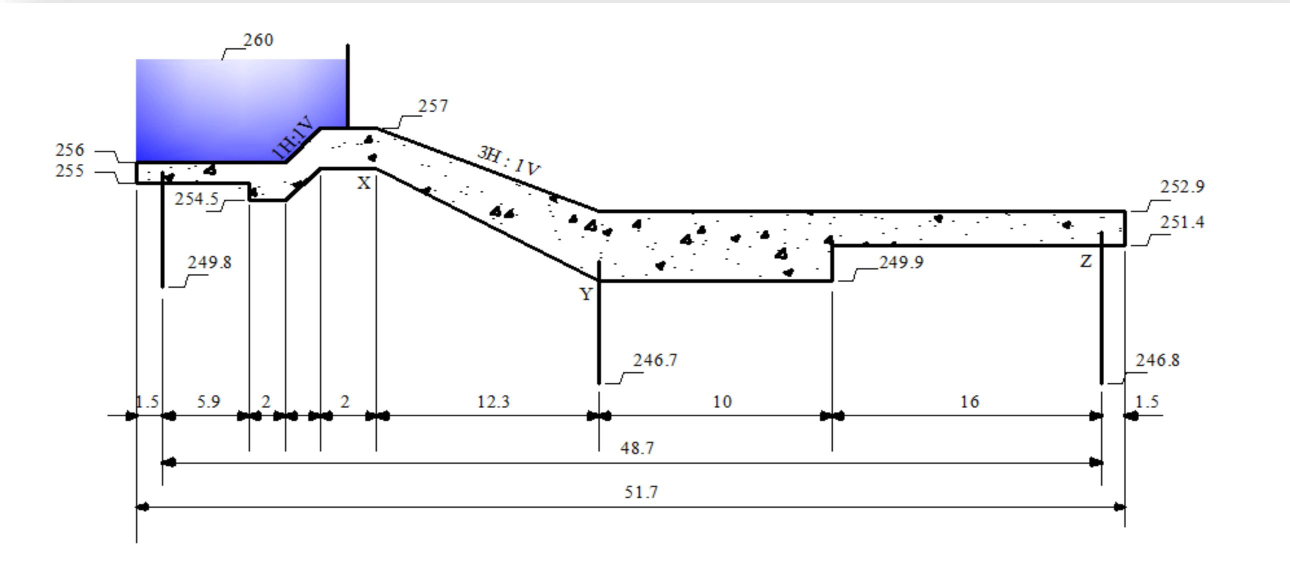


Figure (2.6): Definition Sketch for Example (2.2)

Solution

$$L_w = \frac{1}{3} H + V$$

$N = 1.5+5.9+2+2 + (5.1^2+12.3^2)^{0.5}+10+16+1.5 = 52.2$ m

$V = 1+2\times5.2+0.5 + (0.5^2+0.5^2)^{0.5}+2\times3.2+1.5++2\times4.6+1.5 = 31.2$ m

$$\therefore L_w = \frac{1}{3}\times 52.2 + 31.2 = 48.6 \text{ m}$$

$H = 260-252.9 = 7.1$ m

$H / L_w = 7.1/ 48.6 = 1/ 6.84 < 1/5$ Ok - The structure is safe against piping.

$L_x = 16.4$ m, $L_Y = 20.83$ m, $L_Z = 37.4$ m

$H_X = (7.1/48.6) \times (48.6 - 16.4) = 4.7$ m of water

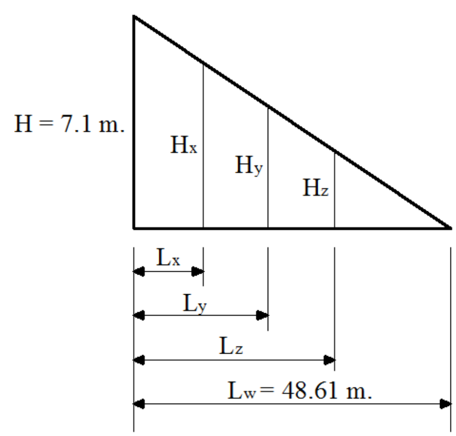
$H_Y = (7.1/48.6) \times (48.6 - 20.83) = 4.05$ m of water

$H_Z = (7.1/48.6) \times (48.6 - 37.4) = 1.63$ m of water

$t_X = 4.7 /(2.4-1) = 3.36$ m of concrete > 2 not OK

$t_Y = 4.05 /(2.4-1) = 2.89$ m of concrete < 3 OK

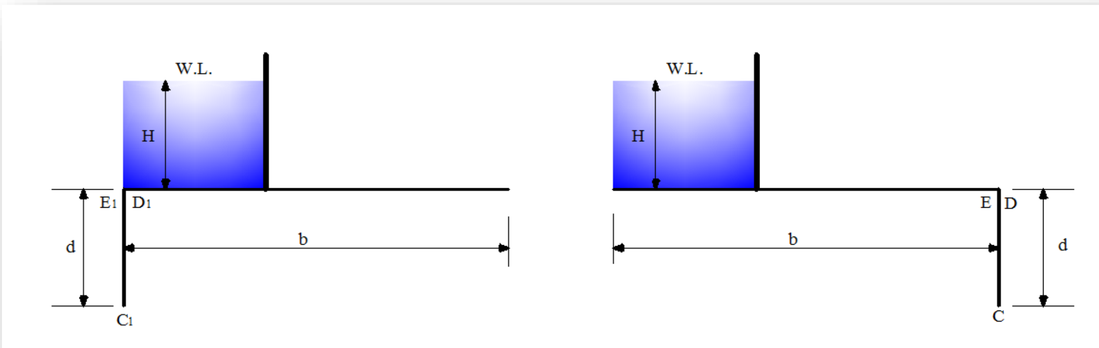
$t_Z = 1.63 /(2.4-1) = 1.16$ m of concrete < 1.5 OK



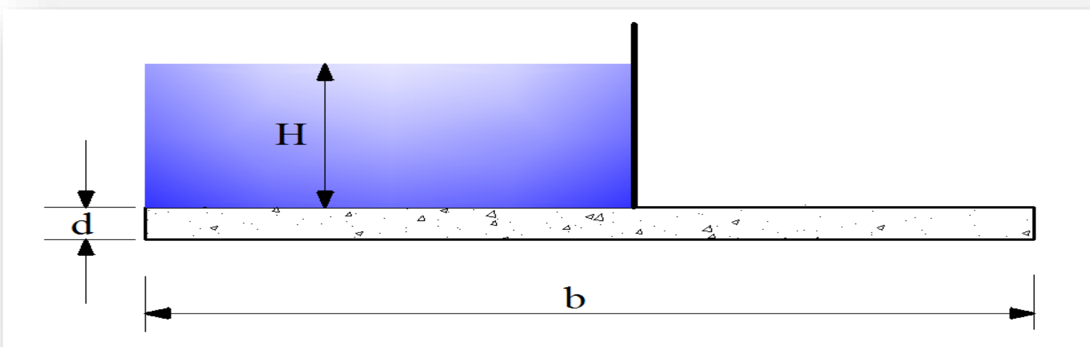
2.3. Khosla's Theory

It is used to find uplift pressure at key points in a barrage or weir. The following specific cases of general form were considered:

a. A straight horizontal floor of negligible thickness with a sheet pile at either ends.



b. A straight horizontal floor depressed below the bed but with no vertical cut off.



c. A straight horizontal floor thickness with a sheet pile at some intermediate position.

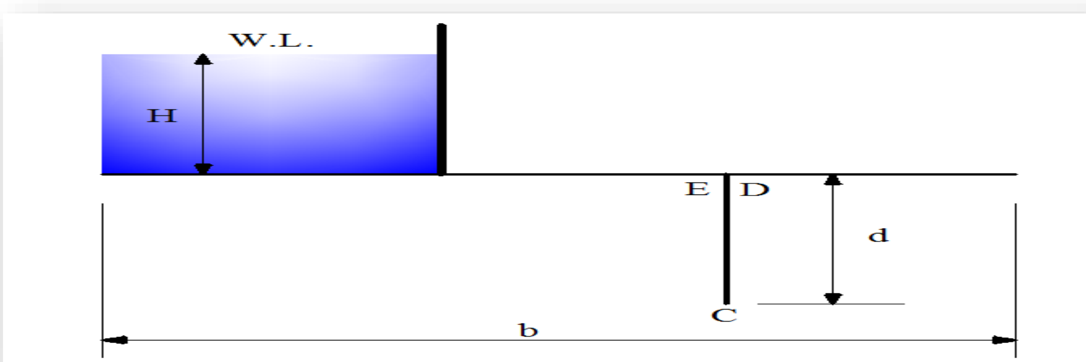


Figure (2.7): Specific Cases of General Form for Hydraulic Structures

Some conclusions related with Khosla's theory:

1. Outer faces of end sheet piles were much more effective than the inner ones.

2. Intermediated piles of smaller length were ineffective except for local redistribution of pressure.
3. Undermining of floor started from tail end. It was absolutely essential to have a reasonably deep vertical cut off at the downstream end to prevent undermining.
4. Khosla and his associates took into account the flow pattern below the impermeable base of hydraulic structure to calculate uplift pressure and exit gradient.
5. Seeping water below a hydraulic structure does not follow the bottom profile of the impervious floor as stated by Bligh but each particle traces its path along a series of streamlines.

The key points are the junctions of the floor and the pile lines on either side, and bottom point of the pile line, and the bottom corners in the case of a depressed floor. The percentage pressures at these key points for the simple forms into which the complex profile has been broken is valid for the complex profile itself, if corrected for: The following corrections are effected:

1- The correction for the mutual interference of piles.

2- Correction for the thickness of floor.

3- Correction for the slope of the floor.

2.3.1. The Correction for the Mutual Interference of Piles

For the figure (2.8) shown below.

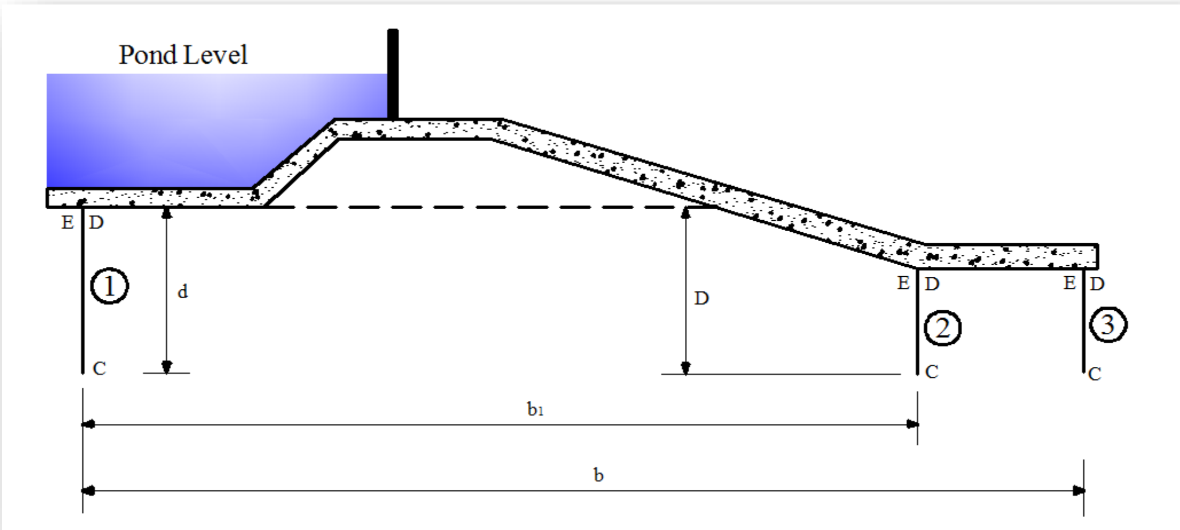


Figure (2.8): Definition Sketch for Mutual Interference between Sheet Piles

$$C_m = \pm 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$$

C_m: correction at the corner (correction in a percentage of head).

b: total floor length.

b': distance between two piles.

d: depth of pile on which the effect of pile is required to be determined.

D: depth of pile whose influence has to be determined on the neighboring pile of depth (d)

Notes:

1- The correction is positive (+ve) for points in the rear of back water and negative (-ve) for points forward in the direction of flow. i.e.

Effective of D/S pile on U/S pile (+ve).

Effective of U/S pile on D/S pile (–ve).

2- This equation does not apply to the effect of an outer pile on an intermediate pile if the latter is equal to or smaller than the former and is at a distance less than twice the length of the outer line.

2.3.2. Correction for the Thickness of Floor

For different locations of piles, the corrections to be applied are as follows:

a- Correction for u/s pile

Corrected pressure at point C_1

$$C_t = +\left(\frac{\Phi D - \Phi C}{d_1}\right) t_1$$

b- Correction for intermediate pile

Corrected pressure at point E_1

$$C_t = -\left(\frac{\Phi E - \Phi D}{d_2}\right) t_2$$

c- Correction for d/s pile

Corrected pressure at point E_1

$$C_t = -\left(\frac{\Phi E - \Phi D}{d_3}\right) t_3$$

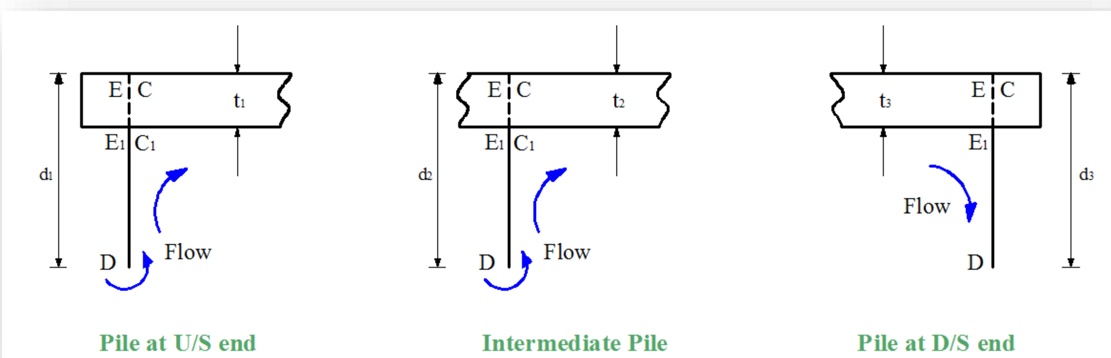


Figure (2.9): Definition Sketch for Floor Thickness Correction at Different Key Points

Where:

C_t : Correction due to thickness of the floor

d_1, d_2, d_3: depth of piles

t_1, t_2, t_3: floor thickness

We can use the following equations to find the uplift pressure (Φ) at E, C &D:

a. U/S & D/S piles

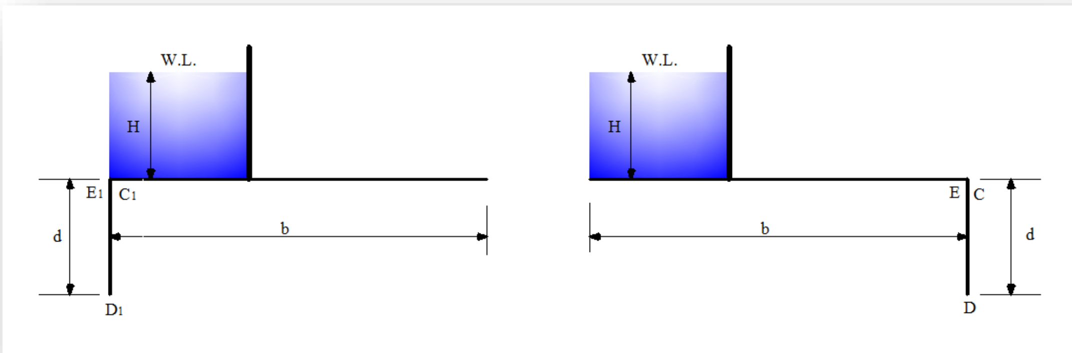


Figure (2.10): Definition Sketch for Uplift Pressure Percentage at U/S and D/S Sheet Piles

$$\Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) \quad , \quad \Phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)$$

$$\Phi_{C1} = 100 - \Phi_E \quad , \quad \Phi_{D1} = 100 - \Phi_D$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \quad ; \quad \alpha = \frac{b}{d}$$

b. Intermediate pile

$$\Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1-1}{\lambda}\right) \quad , \quad \Phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1+1}{\lambda}\right) \quad , \quad \Phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$$\lambda_1 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2} \quad , \quad \lambda = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2}$$

$$\alpha_1 = \frac{b_1}{d} \; ; \; \alpha_2 = \frac{b_2}{d}$$

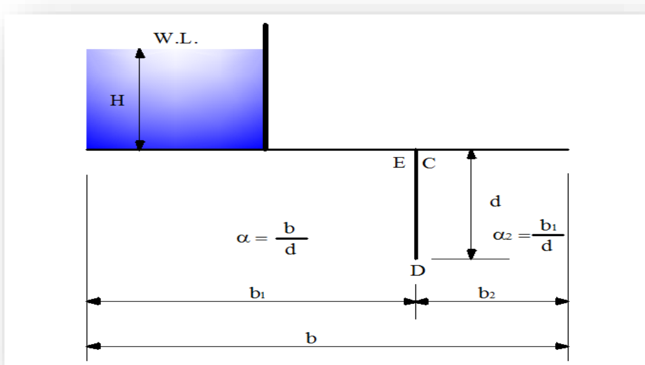


Figure (2.11): Definition Sketch for Uplift Pressure Percentage at Intermediate Sheet Pile

2.3.3. Correction for the Slope of the Floor

Due to sloping floor, a suitable percentage correction is to be applied.

$$C_s = \mp \frac{b}{b_1} C$$

C_s: slope correction.

C: coefficient due to slope from table (3)

b: horizontal length of slope.

b_1: distance between two piles which the sloping floor is located.

Notes:

1- The correction is ***plus*** for the down slopes and ***minus*** for the up slopes following the direction of flow.

2- The slope correction is applicable to the key point of pile line fixed at the ***beginning*** or ***the end*** of the slope.

Table (2.3): Khosla's Theory Slope Corrections

Slope (V:H)	1:1	1:2	1:3	1:4	1:5	1:6	1:7	1:8
% Correction	11.2	6.5	4.5	3.3	2.8	2.5	2.3	2.0

2.3.4. Exit Gradient

Exit gradient at the downstream end of an impervious floor length ***b*** and vertical cutoff d is given by

$$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$$

G_E: exit gradient

H: maximum static head

d: depth of d/s cutoff .

b: length of floor (horizontal).

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \quad , \quad \alpha = \frac{b}{d}$$

Table (2.4): Safe Exit Gradient for Different Type of Soils

Type of soil	Shingle	Course sand	Fine sand
Safe exit gradient (G_E)	$\frac{1}{4}$ to $\frac{1}{5}$	$\frac{1}{5}$ to $\frac{1}{6}$	$\frac{1}{6}$ to $\frac{1}{7}$

Example (2.3): A hydraulic structure with length of horizontal floor in alluvial soil 15m and 3m deep vertical sheet pile is attached at its downstream end and the head of water is 4.0m. Find the thickness of the floor (using Khosla's theory). Is the structure safe against the exit gradient? ($F = 8$, $G = 2.45$).

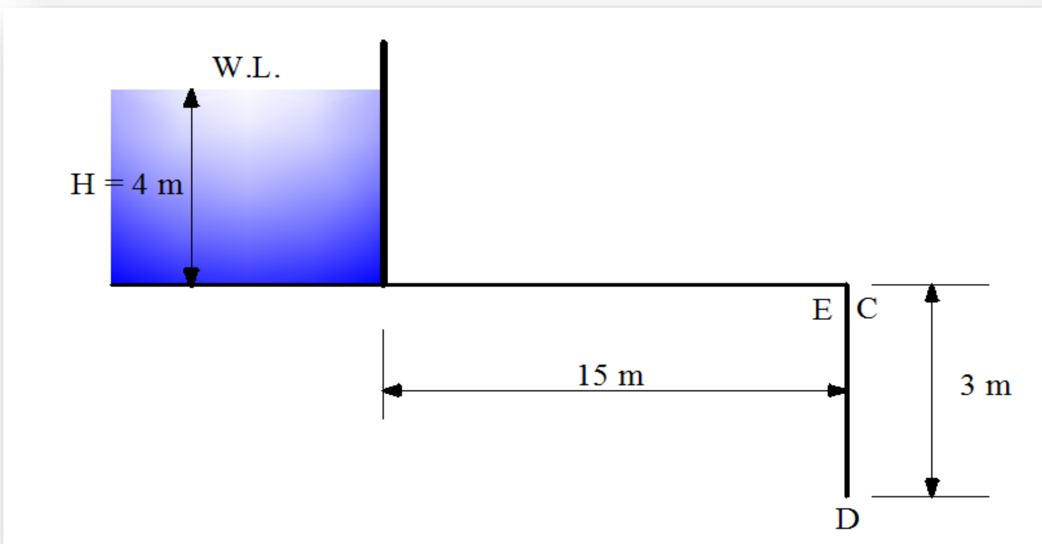


Figure (2.12): Definition Sketch for Example (2.3)

Solution

$$t_E = \frac{H_E}{G-1} \quad , \quad \Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \quad , \quad \alpha = \frac{b}{d} = \frac{15}{3} = 5$$

$$\therefore \lambda = 3.05$$

$$\Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{3.05-1}{3.05}\right) = 0.265$$

$\therefore H_E = 0.265\times4 = 1.06$ m of water

$$t_E = \frac{1.06}{2.45-1} = 0.73 \text{ m of concrete}$$

$$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}} = \frac{4}{3}\frac{1}{\pi\sqrt{3.05}} = 0.243$$

$$F = \frac{1}{G_E} = \frac{1}{0.243} = 4.11 < 8 \text{ the structure is unsafe}$$

H.W. 2: Determine the percentage pressures at various key points in figure below. Also determine the exit gradient and plot the hydraulic gradient line for pond level on U/S and no flow on D/S.

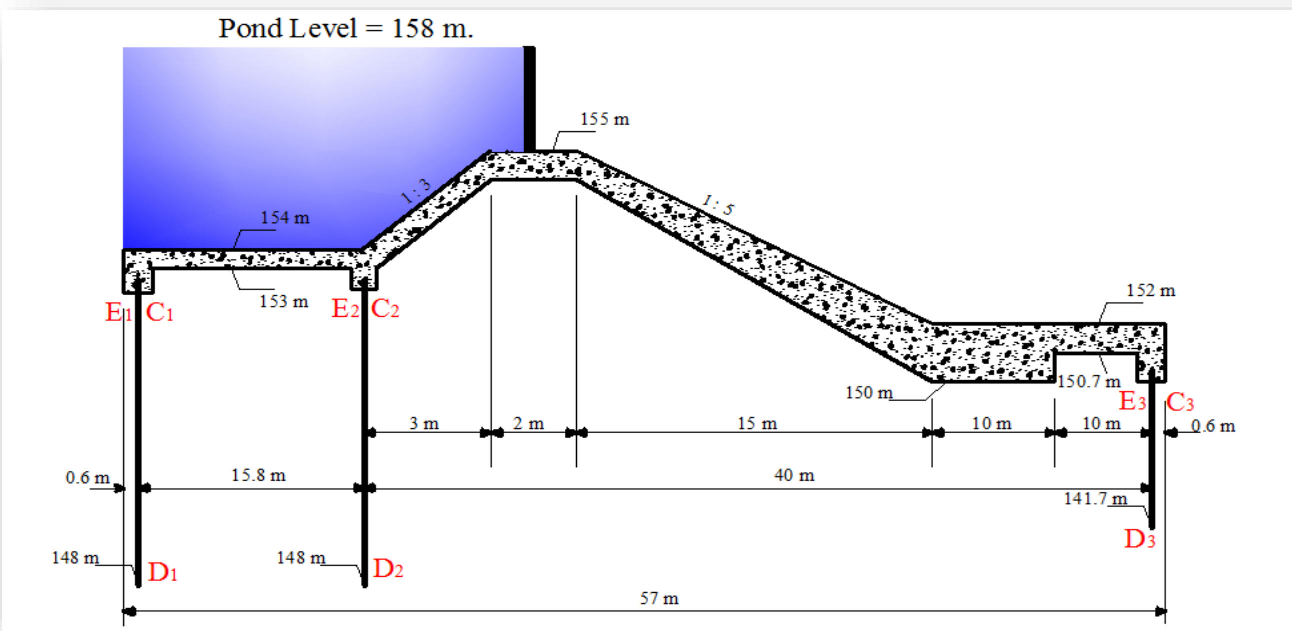


Figure (2.13): Definition Sketch for Homework #2

H.W. 3: Use KhosIa's method to calculate the percentage uplift pressures at the three cut-offs for a barrage foundation profile as shown in the figure below applying corrections as applicable. (Given slope correction for 1 in 4 slope is 3.3%)

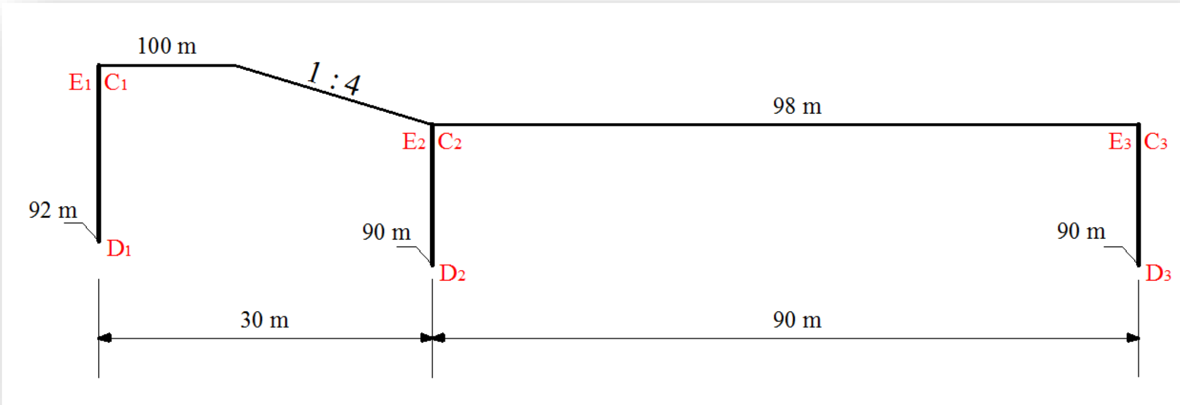


Figure (2.14): Definition Sketch for Homework #3

Chapter Three

Hydraulic Jump & Energy Dissipation Structures

3.1. Introduction

The most common application of the momentum equation in open-channel flow deals with the analysis of the *hydraulic jump*. The rise in water level, which occurs during the transformation of the unstable "rapid" or supercritical flow to the stable "tranquil" or subcritical flow, is called hydraulic jump, manifesting itself as a standing wave. At the place, where the hydraulic jump occurs, a lot of energy of the flowing liquid is dissipated (mainly into heat energy). This hydraulic jump is said to be a dissipater of the surplus energy of the water. Beyond the hydraulic jump, the water flows with a greater depth, and therefore with a less velocity.

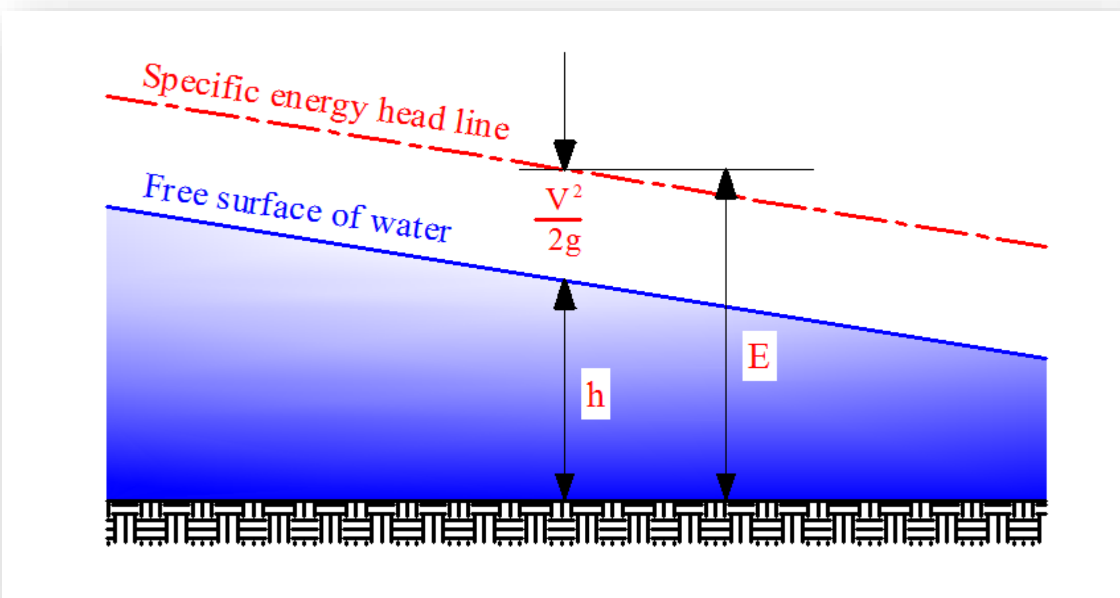


Figure (3.1): Total Energy Head in Open Channel Flow

The hydraulic jump has many practical and useful applications. Among them are the following:

1. Reduction of the energy and velocity downstream of a dam or chute in order to minimize and control erosion of the channel bed.
2. Raising of the downstream water level in irrigation channels.
3. Acting as a mixing device for the addition and mixing of chemicals in industrial and water and wastewater treatment plants. In natural channels the hydraulic jump is also used to provide aeration of the water for pollution control purposes.

The specific energy head, E, of a flowing liquid is defined as the energy head with respect to a datum plane, for instance passing through the bottom of the channel as shown in figure (3.2):

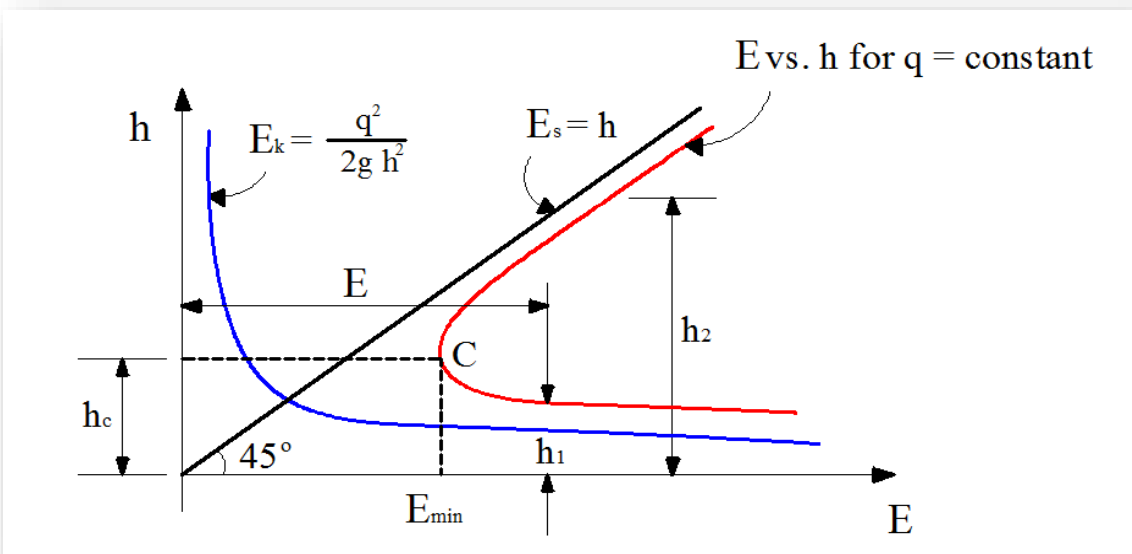


Figure (3.2): The Specific Energy – Head Diagram

Mathematically, the specific energy head reads as:

$$\mathbf{E = h + \frac{V^2}{2g} = E_s + E_k} \qquad \text{…. (3-1)}$$

Where

h = depth of liquid flow

V = mean velocity of liquid

E_s = h = static energy head (also known as potential energy head)

$E_k = \frac{V^2}{2g} = \frac{q^2}{2gh^2}$ = kinetic energy head (depth averaged) and q is the discharge per unit width.

Plotting the specific energy diagram for a channel (water depth h along the vertical axis), may conveniently be done by first drawing the two (independent) curves for static energy and kinetic energy and then adding the respective ordinates. The results is the required *specific energy head curve*.

3.2. Critical Depth and Critical Velocity

We can see in the specific-energy diagram (as shown in figure above) that the specific energy is minimum at point C. The depth of water in a channel, corresponding to the minimum specific energy (as at C in this case) is known as *critical depth*. This depth can be found by differentiating the specific-energy head equation and equating the result to zero. Or,

$$\frac{dE}{dh} = 0$$

Or, substituting $E = h + \frac{V^2}{2g}$, we have:

$$\frac{d}{dh}\left(h + \frac{V^2}{2g}\right) = 0$$

With $V = \frac{q}{h}$, where q is the constant discharge per unit width

$$\frac{d}{dh}\left(h + \frac{q^2}{2gh^2}\right) = 0 \longrightarrow 1 - \frac{q^2}{gh^3} = 0$$

$$\therefore h = \frac{V^2}{g}$$

Since the flow is (assumed to be) critical, the subcript c is added; therefore

$$\therefore h_c = \frac{{V_c}^2}{g}$$

Where

h_c = critical depth and V_c = critical velocity

Replacing h by of hc and V by Vc in the specific-energy head equation, the minimum specific-energy head can be written as:

$E_{min} = h_c + \frac{{V_c}^2}{2g} = h_c + \frac{h_c * g}{2g} = h_c + \frac{h_c}{2} = \frac{3}{2} h_c$

Or the static energy head becomes:

$h_c = \frac{2}{3} E_{min}$

And the kinetic energy head:

$E_{kc} = \frac{{V_c}^2}{2g} = E_{min} - \frac{2}{3} E_{min} = \frac{1}{3} E_{min}$

$h_c = \frac{{V_c}^2}{g} = \frac{(\frac{q}{h_c})^2}{g}$

Or : ${h_c}^3 = \frac{q^2}{g} = (\frac{q^2}{g})^{\frac{1}{3}}$

This is the equation for the critical depth, when the discharge per unit width through the channel is given. Thus, the critical velocity corresponding to the depth of the channel is:

$V_c = \frac{q}{h_c}$

Example 3.1: A channel, 6 m wide, is discharging 20 m^3/s of water. Determine the critical depth and critical velocity, i.e. when the specific energy of the flowing water is minimum.

Solution:

Given: Q = 20 m^3/s , b = 6 m , q = Q/b = 3.33 m^2/s , Q = 3.33 m2/s

Depth of water at minimum specific energy or critical depth:

$h_c = (\frac{q^2}{g})^{\frac{1}{3}} = 1.04$ m

$V_c = \frac{q}{h_c} = 3.2$ m/s

3.3. Types of Flows

Depending on the critical depth as well as the real, occurring depth of water in a channel, three types of flow can be distinguished:

1. Tranquil flow: If the depth of water, in the channel is *greater* than the critical depth, the flow is called tranquil or *subcritical*.

2. Critical flow: If the depth of water in the channel is *critical*, the flow is called *critical*.

3. Rapid flow: If the depth of water in the channel is *smaller* than the critical depth, the flow is called *supercritical*.

Example 3.2: **A channel of rectangular section, 7.5 m wide, is discharging water at a rate of 12 m3/s with an average velocity of 1.5 m/s. Find:**
(a) Specific-energy head of the flowing water,
(b) Depth of water, when specific energy is minimum,
(c) Velocity of water, when specific energy is minimum,
(d) Minimum specific-energy head of the flowing water,
(e) Type of flow.

Solution:

Given: b = 7.5 m , Q = 12 m^3/s , q = 1.6 m^2/s , V = 1.5 m/s , h = q/V = 1.067 m
$E = h + \frac{V^2}{2g} = 1.182$ m, $h_c = (\frac{q^2}{g})^{\frac{1}{3}} = 0.639$ m, $V_c = \frac{q}{h_c} = 2.5$ m/s, $E_{min} = h_c + \frac{{V_c}^2}{2g} = 0.958$ m

Since the depth of water (1.067 m) is larger than the critical depth (0.639 m), the flow is tranquil or subcritical.

3.4. Water Rise in Hydraulic Jump

We can see in the specific-energy diagram (Fig. 3.1) that for a given specific energy E, there are two possible depths h1 and h_2. The depth h_1 is smaller than the critical depth, and h_2 is greater than the critical depth. To calculate these depths, consider two sections, on the upstream and downstream side of a jump, as shown in figure (3.3):

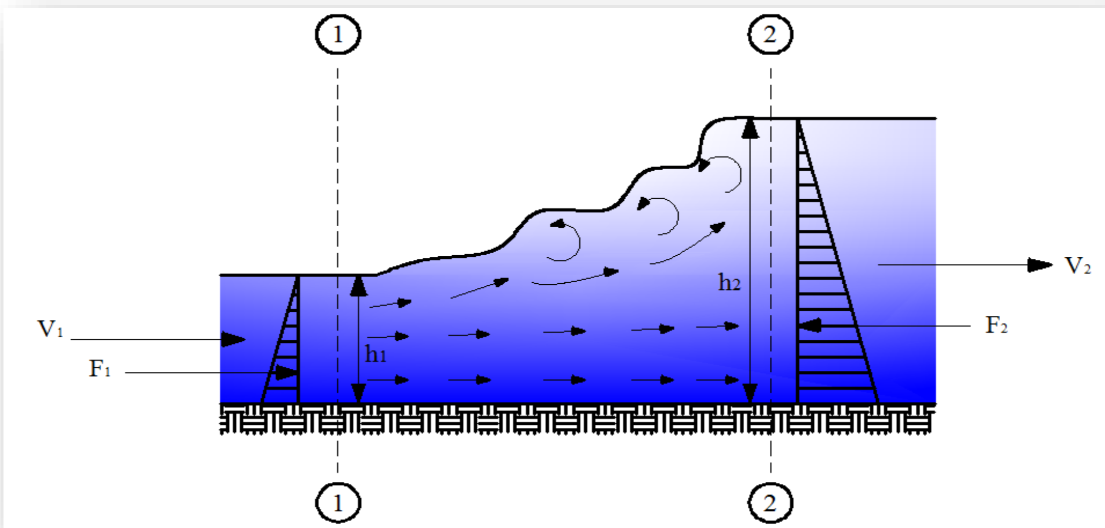


Figure (3.3): Hydraulic Jump Formation

Let 1 - 1 = section on the upstream side of the hydraulic jump,

2 - 2 = section on the downstream side of the hydraulic jump,

h_1, h_2 = depth of flow at section 1 – 1 and 2-2 respectively

V_1, V_2 = flow velocity at section 1 – 1 and 2-2 respectively

q = discharge per unit width = Q/b,

Where Q = total discharge and b = width of channel and hydraulic jump

$q = h_1V_1 = h_2V_2$

Now consider the control volume of water between the sections 1-1 and 2-2, and apply the law of conservation of momentum. Force F_1 on section 1-1:

$$F_1 = \gamma(h_1 * 1)\frac{h_1}{2} = \frac{\gamma h_1^2}{2}$$

Where $\gamma = \rho g$ is the specific weight of the water. Similarly:

$F_2 = \frac{\gamma h_2^2}{2}$

The net force

$F = F_1 - F_2 = \frac{\gamma h_1^2}{2} - \frac{\gamma h_2^2}{2} = \frac{\gamma}{2}\ (h_1^2 - h_2^2)$

This force is responsible for change of velocity from V_1 to V_2

We know that this force is also equal to the change of momentum of the control volume:

Force = mass of water flowing per second × change of velocity

$F = \frac{\gamma q}{g}(V_2 - V_1)$

$\frac{\gamma q}{g}(V_2 - V_1) = \frac{\gamma}{2}\ (h_1^2 - h_2^2)$

$(h_1^2 - h_2^2) = \frac{2q}{g}(V_2 - V_1)\ \frac{2q}{g}\left(\frac{q}{h_2} - \frac{q}{h_1}\right) = \frac{2q^2}{g}\ (\frac{h_1 - h_2}{h_1 h_2})$

Or $(h_1 + h_2)(h_1 - h_2) = \frac{2q^2}{g}\ (\frac{h_1 - h_2}{h_1 h_2})$

$h_1 + h_2 = \frac{2q^2}{g\, h_1 h_2}$

$h_2^2 + h_1 h_2 = \frac{2q^2}{g\, h_1}$

Or $h_2^2 + h_1 h_2 - \frac{2q^2}{g\, h_1} = 0$

Solving the above quadratic equation for h_2, we get:

$$h_2 = -\frac{h_1}{2} \pm \sqrt{\frac{h_1^2}{4} + \frac{2q^2}{g h_1}}$$

Taking only + sign and substituting $q = h_1 V_1$:

$$h_2 = -\frac{h_1}{2} + \sqrt{\frac{h_1^2}{4} + \frac{2 h_1 V_1^2}{g}}$$

The depth of the hydraulic jump or the height of the standing wave is $(h_2 - h_1)$.

Example 3.3: A discharge of 1000 l/s flows along a rectangular channel, 1.5 m wide. What would be the critical depth in the channel? If a standing wave is to be formed at a point, where the upstream depth is 180 mm, what would be the rise in the water level?

Solution:

Q = 1000 l/s = 1 m^3/s, b = 1.5 m, h_1= 180 mm, q = 0.67 m^2/s, $h_c = (\frac{q^2}{g})^{\frac{1}{3}}$ = 0.358 m

Let h_2 be the depth of the flow on the downstream side of the hydraulic jump.

$h_2 = -\frac{h_1}{2} + \sqrt{\frac{h_1^2}{4} + \frac{2h_1V_1^2}{g}}$ = 0.63 m = 630 mm

Rise in water level $\Delta h = h_2 - h_1$ = 450 mm

3.5. Energy Loss due to Hydraulic Jump

The loss of energy head due to the occurrence of the hydraulic jump is the difference between the specific-energy heads at sections 1-2 and 2-2. Mathematically,

$$\Delta E = (h_1 + \frac{V_1^2}{2g}) - (h_2 + \frac{V_2^2}{2g})$$

The energy-head loss in the jump is then equal to $E_1 - E_2$, represented by ΔE. After some elaboration it can be derived:

$$\Delta E = E_1 - E_2 = \frac{(h_2 - h_1)^3}{4\, h_1 h_2}$$

Example 3.4: A rectangular channel, 6 m wide, discharges 1200 l/s of water into a 6 m wide apron, with zero slope, with a mean velocity of 6 m/s. What is the height of the jump? How much power is absorbed in the jump?

Solution:

b = 6 m , Q = 1200 l/s = 1.2 m^3/s , V = 6 m/s , q = 1.2/6 = 0.2 m^2/s

$h_c = (\frac{q^2}{g})^{\frac{1}{3}}$ = 0.16 m

$V_{1c} = \frac{q}{h_c} = 1.25$ m/s $< V_1$ $\longrightarrow$ $\therefore$ *supercritical flow (Occurence of hydraulic jump)*

Height of hydraulic jump

$h_1 = Q / (V_1 * b) = 0.033$ m

$$h_2 = -\frac{h_1}{2} + \sqrt{\frac{h_1^2}{4} + \frac{2h_1V_1^2}{g}} = 0.476 \text{ m}$$

$\Delta h = h_2 - h_1 = 0.443$ m

Energy absorbed in the jump

Drop of specific energy head:

$\Delta E = E_1 - E_2$

We know that due to the continuity of the discharge:

$V_1 h_1 = V_2 h_2$ $\longrightarrow$ $\therefore$ $V_2 = (v_1 h_1 / h_2) = 0.42$ m/s

$$E_1 - E_2 = \left(h_1 + \frac{V_1{}^2}{2g}\right) - \left(h_2 + \frac{V_2{}^2}{2g}\right) = 1.384 \text{ m}$$

Dissipation of power in hydraulic jump

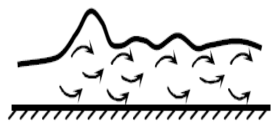
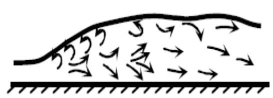
$\Delta P = \rho g Q (E_1 - E_2) = 16.3$ kW

3.6. Classification of Hydraulic Jump

Hydraulic jumps on a horizontal bottom can occur in several distinct forms. Based on the Froude number of the supercritical flow directly upstream of the hydraulic jump, several types can be distinguished (see Table 3.1).

It should be noted that the ranges of the Froude number given in Table 3.1 for the various types of jump are not clear-cut but overlap to a certain extent depending on local conditions.

Table (3.1): Classification of Hydraulic Jump

Type of Jump	Froude Number	Illustration	Description
Critical Flow	Fr = 1	No Jump	No Jump
Undular Jump	1 < Fr < 1.7		The water surface shows undulations
Weak Jump	1.7 < Fr < 2.5		A series of small rollers develop on the surface of the jump, but the downstream water surface remains smooth. The velocity throughout is fairly uniform, and the energy loss is low
Oscillating Jump	2.5 < Fr < 4.5		There is an oscillating jet entering the jump from bottom to surface and back again with no periodicity. Each oscillation produces a large wave of irregular period which, very commonly in canals, can travel for meters doing unlimited damage to earthen banks and rip-raps
Steady Jump	4.5 < Fr < 9		The downstream extremity of the surface roller and the point at which the high velocity jet tends to leave the flow occur at practically the same vertical section. The action and position of this jump are least sensitive to variation in tailwater depth. The jump is well-balanced and the performance is at its best. The energy dissipation ranges from 45 to 70%
Strong Jump	Fr > 9		The high-velocity jet grabs intermittent slugs of water rolling down the front face of the jump, generating waves downstream, and a rough surface can prevail. The jump action is rough but effective since the energy dissipation may reach 85%

3.8. The Initial Depth and the Sequent Depth

It may be noted that the depth before the jump is always less than the depth after the jump. The depth before the jump is called the *initial depth* h_1 and that after the jump is called the *sequent depth* h_2. The initial and sequent depths h_1 and h_2 are shown on the specific-energy head curve (Fig. 3.7). They should be distinguished from the alternative depths h_1 and h_2, which are the two possible depths for the same specific energy. The initial and sequent depths are the actual depths before and after a jump. The specific energy head E_1 at the initial depth h_1 is greater than the specific-energy head E_2 at the sequent depth h_2 by an amount equal to the energy loss ΔE. If there were no energy losses, the initial and sequent depths would become identical with the alternative depths (in a prismatic channel).

We can determine a relationship between the initial depth and the sequent depth of a hydraulic jump on a horizontal floor in a rectangular channel as shown in the following formula:

$$\frac{h_1}{h_2} = \frac{1}{2}\left(\sqrt{1 + 8\,{Fr_2}^2} - 1\right) \qquad \text{With} \qquad Fr_2 = \frac{V_2}{\sqrt{g\,h_2}}$$

Example 3.5: A vertical sluice gate with an opening of 0.67 m produces a downstream jet with a depth of 0.40 m when installed in a long rectangular channel, 5.0 m wide, conveying a steady discharge of 20 m3/s. It is assumed that the flow downstream of the gate eventually returns to a uniform flow depth of 2.5 m.

(a) Verify that a hydraulic jump occurs.

(b) Calculate the energy-head loss in the jump.

(c) If the energy-head loss through the gate is ($0.05\frac{{V_{II}}^2}{2g}$), calculate the depth upstream of the gate and the force on the gate.

Solution:

Given: gate opening: h_o = 0.67 m downstream jet depth: h_{II} = 0.40 m

Channel wide: W = 5.0 m discharge: Q = 20 m^3/s

Sequent depth: h_2 = 2.5 m

Jump occurs? Energy head loss ΔE? Upstream depth h_I? Force on the gate?

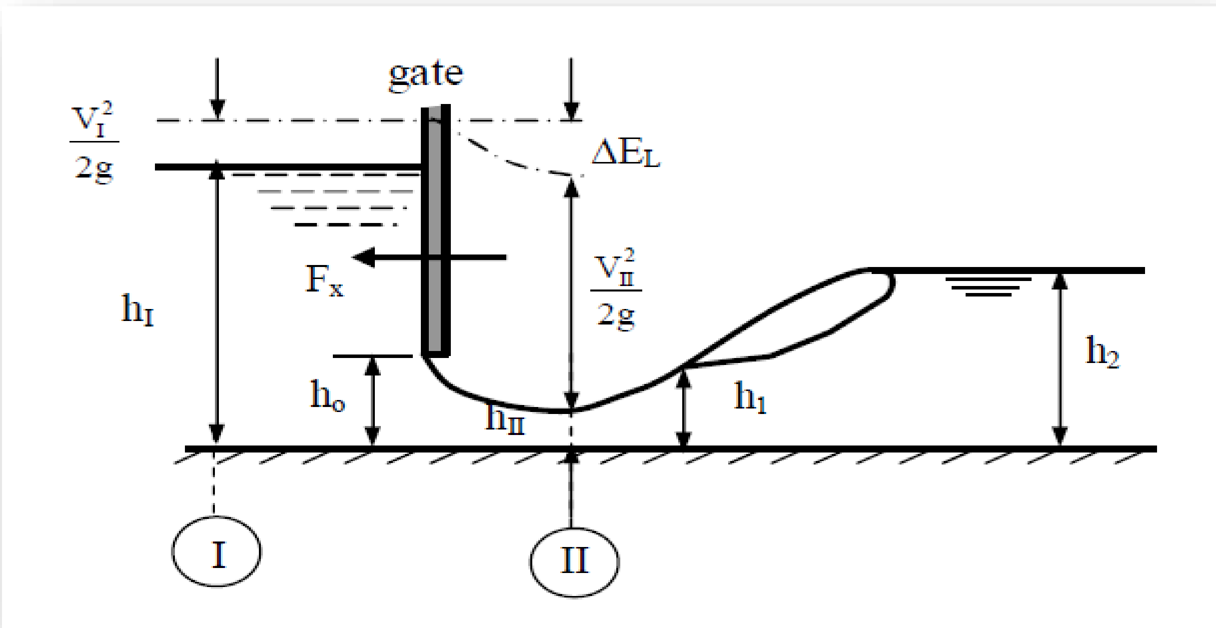


Figure (3.4): General Description of Flow Conditions for Example (3.5)

(a) If a hydraulic jump is to form, the required initial depth, h_1, must be greater than the jet depth, h_{II}. Velocity of flow in the downstream section:

$V_2 = Q/Wh_2$ = 1.6 m/s

$$Fr_2 = \frac{V_2}{\sqrt{g\,h_2}} = 0.323$$

Initial depth: $h_1 = \frac{h_2}{2}\left(\sqrt{1 + 8\,Fr^2} - 1\right)$ = 0.443 m

Because $h_1 > h_{II}$, therefore a jump will form

b) Apply the energy equation from section I to II:

$$(h_1 + \frac{V_1{}^2}{2g}) = (h_{II} + \frac{V_{II}{}^2}{2g}) + 0.05\frac{V_{II}{}^2}{2g}$$

$V_1 = Q/Wh_1 = 4/h_1$ and $V_{II} = Q/Wh_{II} = 10$ m/s, so $\frac{V_{II}{}^2}{2g} = 5.097$ m

Whence $h_I = 5.73$ m

Let F_x the gate reaction per unit width.

Apply the momentum equation to the control volume between section I and section II:

$$\mathbf{F_x = \frac{\rho g\, h_I^2}{2} - \frac{\rho g\, h_{II}^2}{2} + \rho(V_I^2 h_I - V_{II}^2 h_{II})}$$

(Note that the force due to the friction head loss through the gate is implicitly included in the above equation since this effects the value of h_I)

Whence $F_x = 123$ kN/m

--

3.9. Stilling Basins

When water is released over the spillway, the potential energy is converted into kinetic energy at the base of spillway. This energy must be dissipated in order to prevent the possibility of sever scouring of downstream riverbed and the undermining of foundation which may cause failure of spillway and dam. For this purpose energy dissipaters must be used which perform the energy reduction by converting the kinetic energy into turbulence and finally into heat. The dissipation of energy can be achieved by means of several methods such as stilling basins. The formation of hydraulic jump in the stilling basin will lead to dissipation of excess energy. In the stilling basin, the exiting supercritical flow from the spillway is reduced to subcritical flow by a hydraulic jump.

Optimum energy dissipation will occur when the flow enters the dissipater uniformly. To ensure that a stilling basin performs its function efficiently (i.e. dissipation of energy is occurred properly), basin should be designed in such a way that

the elevation of tail water depth in the downstream channel not be much less than the elevation of conjugate depth of jump.

Design of stilling basin is governed by several parameters such as:

1) Nature of foundation,
2) Approach Froude number,
3) Impact angle of flow with respect to stilling basin floor,
4) Tail water level and
5) Economical considerations.

In order to reduce the cost of project, certain components, such as baffle blocks, end sills and chute blocks are installed along the basin floor to control and stabilize the jump which helps to the dissipation of excess energy (Fig. 1). The use of these devices permits the shortening of the basin and acts as a safety factor against sweep out of the jump.

3.10. Basic Elements of Stilling Basin

The basins are usually provided with the following special appurtenances:

Chute blocks: Are used to form a serrated device at the entrance to the stilling basin. *Their function* is to furrow the incoming jet and lift a portion of it from the floor producing a shorter length of jump than would be possible without them.

Baffles: Are blocks placed in the intermediate position across the basin floor. *Their function* is to dissipate energy mostly by impact action. They are useful in small structures with low incoming velocities. They are unsuitable where high velocities make cavitation possible.

The sill: Is usually provided at the end of stilling basin. *Its function* is to reduce further the length of the jump and to control scour. The sill has additional function of

diffusing the residual portion of high velocity jet that may reach the end of the basin.

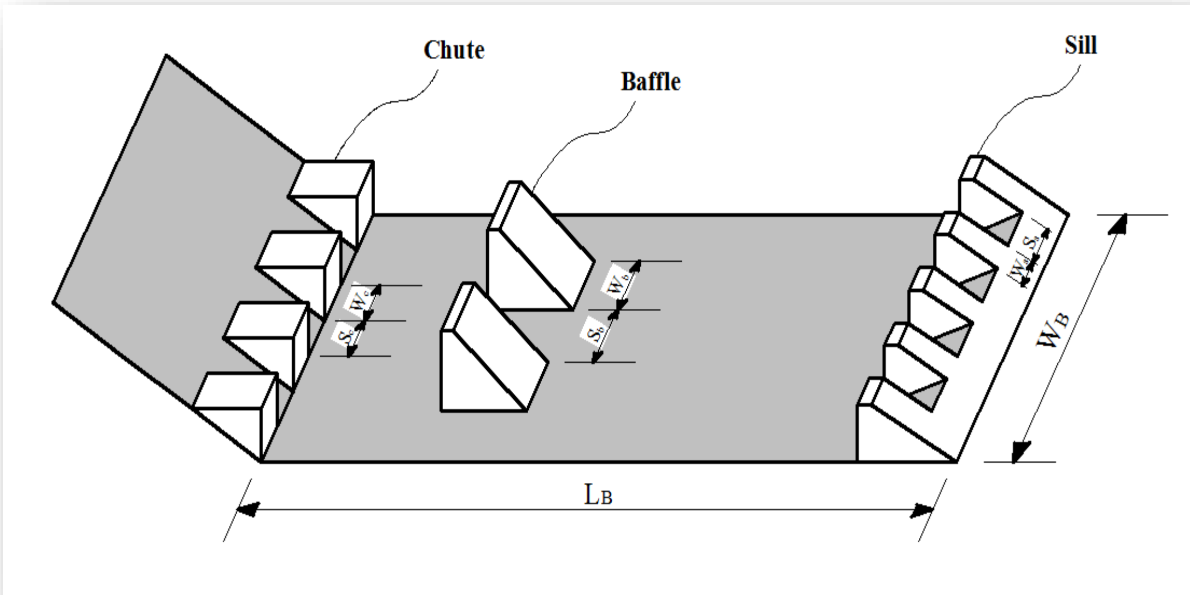


Figure (3.5): Typical Sketch for a Stilling Basin

3.11. Classification of Stilling Basins

Stilling basins can be classified into:

1. Stilling basins in which F_r < **4.5**. This is generally encountered on weirs and barrages.
2. Stilling basins in which F_r > **4.5**. This is a general feature for medium and high dams.

a. Stilling Basin Design for Low Froude Numbers Fr < 4.5

a.1. R.S. Varshney Stilling Basin

On the basis of extensive model test, R. S. Varshney has evolved a stilling basin design, which is exhibited in figure (3.8).

a.2. S.A.F. (Saint Anthony Falls) Stilling Basin

This stilling basin was developed at the St. Anthony Falls Hydraulic Laboratory, University of Minnesota. The design rules are:

1- The stilling basin can be designed for Froude numbers F_1=1.7 and 17, Length of basin(L_B):

$$L_B = \frac{4.5D_2}{F_r^{0.76}}$$

2- The height of chute blocks and floor blocks= D_1 and the width and spacing = 0.75 D_1.

3- The distance from chute blocks to floor blocks= $L_B/3$

4- No floor blocks should be placed closer to side wall than $3D_1/8$

5- The floor blocks should be placed D from the openings between chute blocks.

6- The floor blocks should occupy between 40-55% of basin width.

7- The height of end sill $C = 0.07\ D_2$

8- Tailwater depth above stilling basin floor is given by

A. For F_r = 1.7 to 5.5

$$T_w = \left(1.1 - \frac{F_1^2}{120}\right) D_2$$

B. For F_r = 5.5 to 11

$$T_w = 0.85\ D_2$$

C. For F_r = 11 to 17

$$T_w = \left(1.1 - \frac{F_1^2}{800}\right) D_2$$

9- Height of side wall above tailwater $Z = D_2/3$.

10- Wing walls should be equal in height to stilling basin side walls. The top of the wing wall should have a slope of 1:1.

11- The wing wall should be placed at an angle 45º to the outlet centerline.

12- Cutoff wall of normal depth should be used at the end of stilling basin.

13- The effect of entrained air should be neglected in the design of stilling basin.

a.3. U/SB.R. Stilling Basin IV

This stilling basin is recommended for $F_r = 2.5\text{-}4.5$ which usually occur on canal structures and diversion dams. For this range of Froude number an oscillating jump will be produced in the stilling basin generating a wave that is difficult to dampen. U/SB.R. Stilling Basin IV is designed to solve this problem. For better performance, it is desirable to make the blocks narrower than D_1 preferable 0.75 D_1 and to set the tail water (T_w) = (1.05-1.1) D_2. The length of basin equals the length of the jump. Basin IV is applicable to rectangular cross-sections only. All details are shown in figure (3.6).

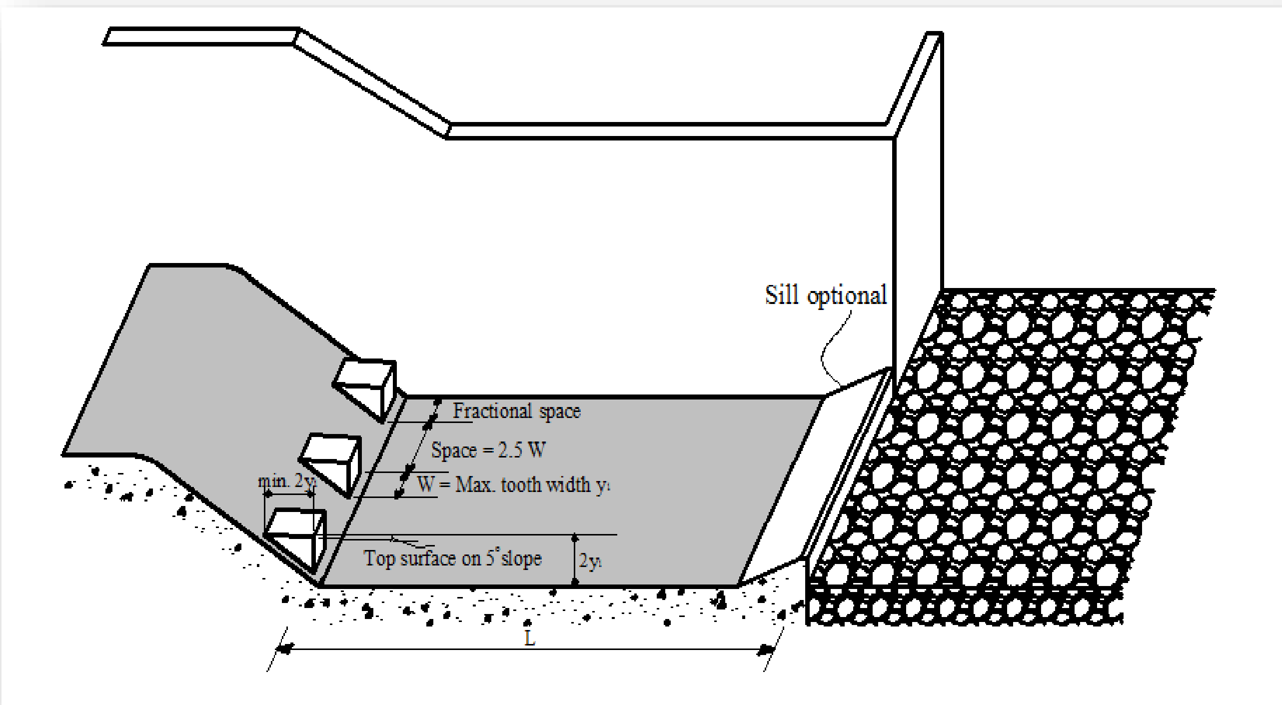


Figure (3.6): Typical Sketch for a Stilling Basin IV

a.4. Indian Standard Stilling Basin

Definition sketches are given below in figure (3.7), (by Varshney).

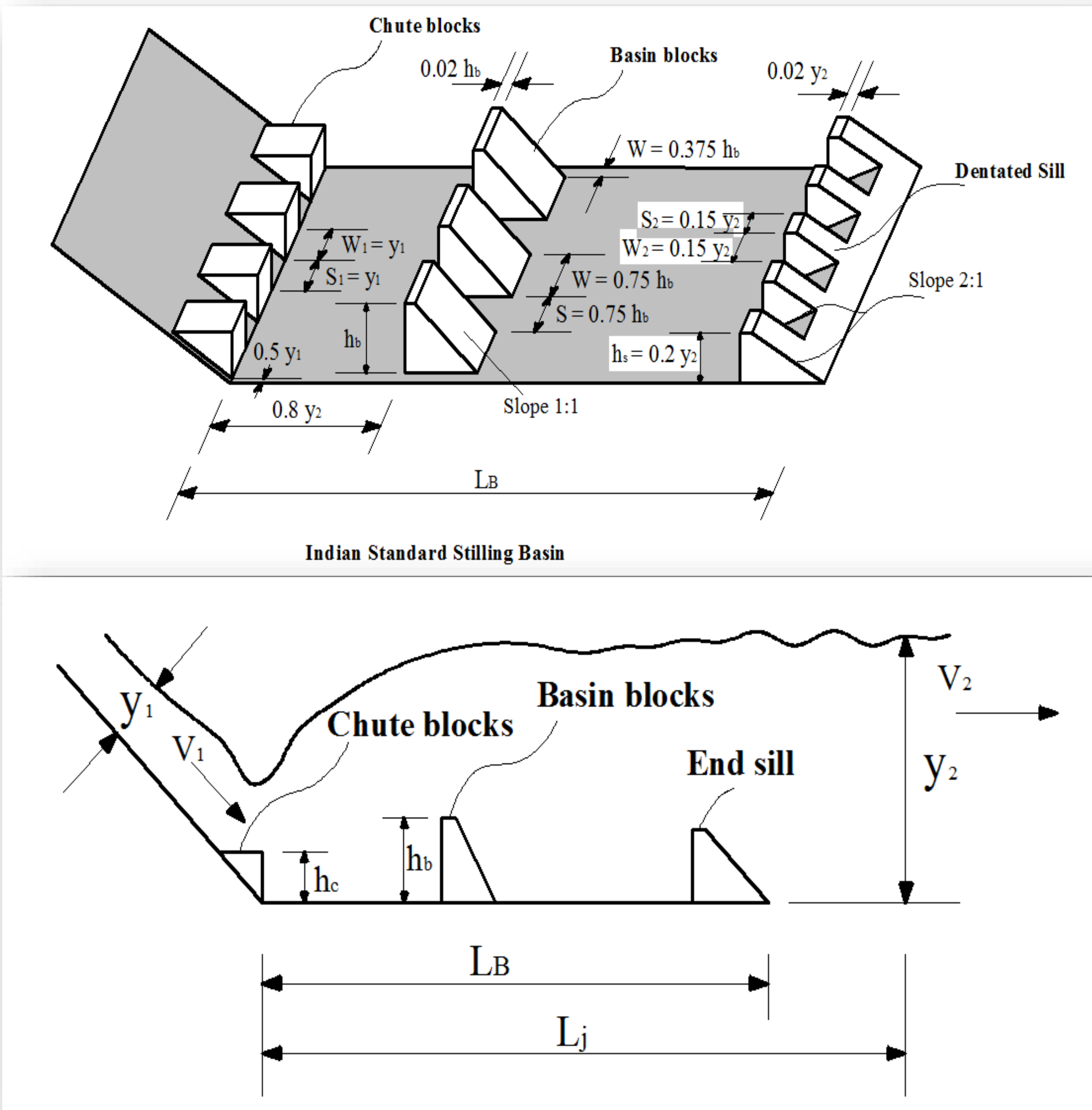
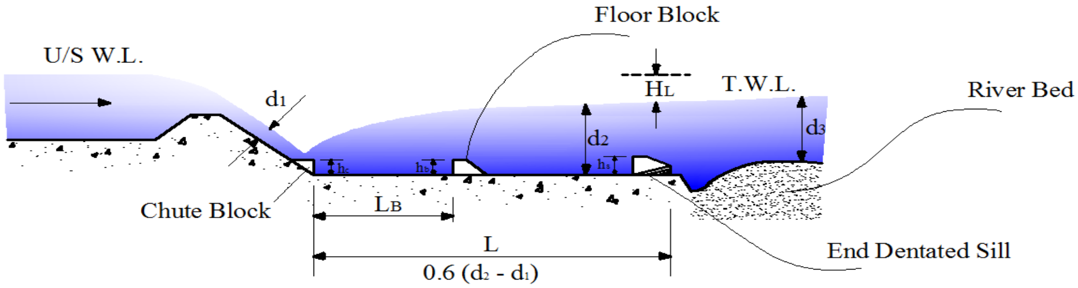
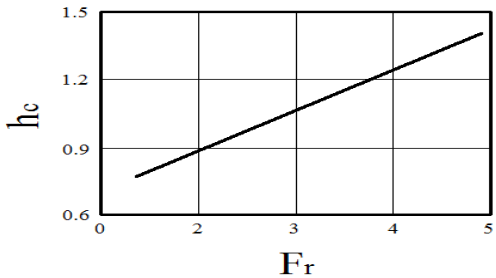
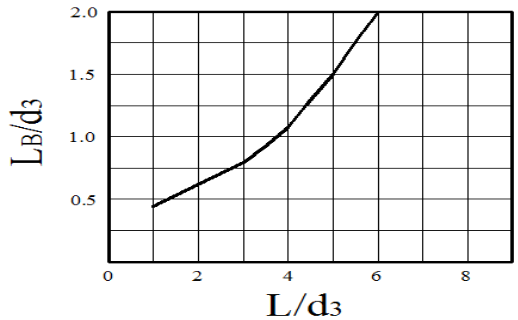


Figure (3.7): Typical Sketch for Indian Stilling Basin

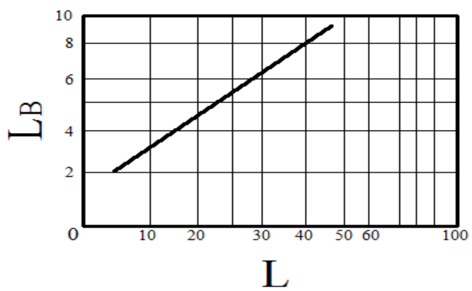




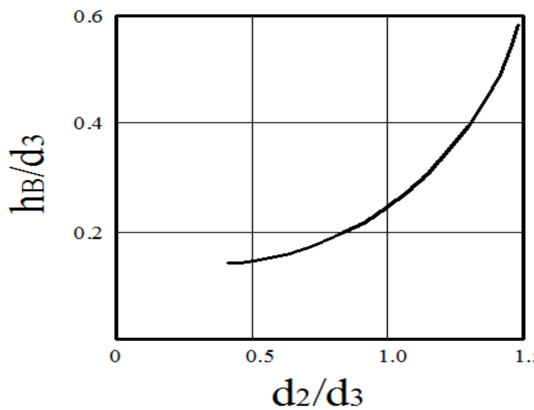
Height of chute blocks in terms of Froude number



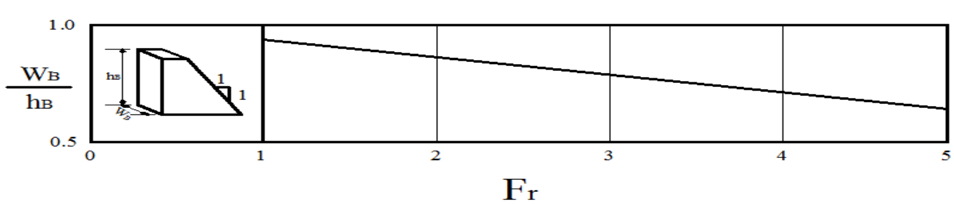
Relation between length of stilling basin and position of floor blocks



Relation between length of stilling basin and position of floor blocks

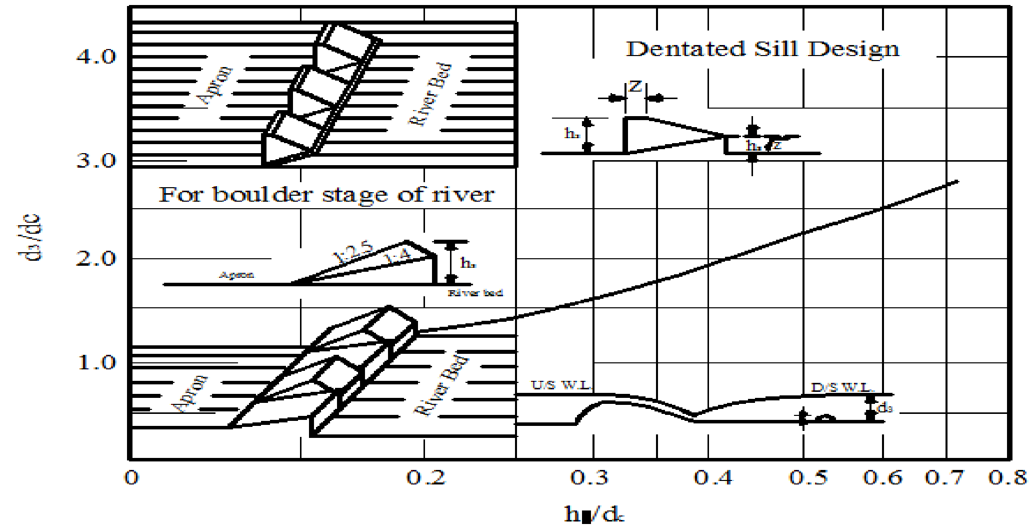


Height of floor blocks in relation to d_2 depth

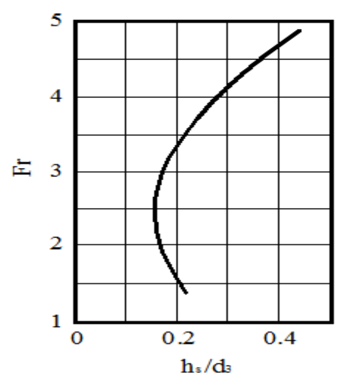


Relation of ratio of width to height of floor blocks and Froude number

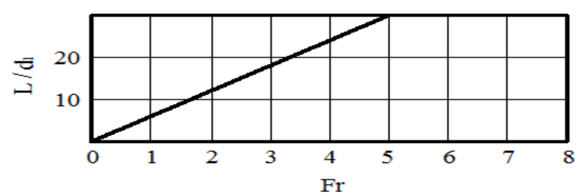
Figure (3.8): Continued



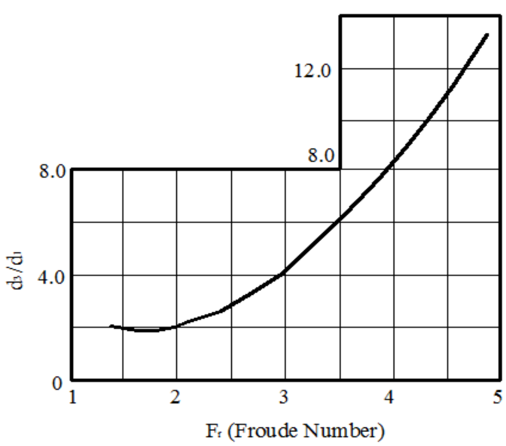
Variation of sill height with normal river depth



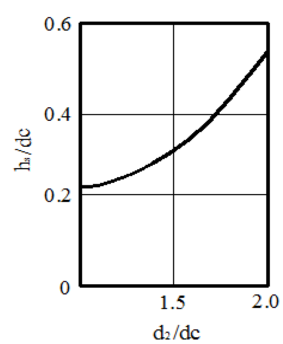
Variation of sill height with Fr



Length of stilling basin in terms of Fr and d1



Relation between Fr and d1



Height of floor block in relation to d2 depth

Figure (3.8): Relations between Hydraulic and Geometric Parameters for Varshney Stilling Basin

b. Stilling Basin Design for Froude Numbers Fr > 4.5

b.1. S.A.F. Stilling Basin.

b.2. Indian Standard Stilling Basin II: The dimension sketches are given in figure (3.7), Varshney.

b.3. USBR Stilling Basin II: This design is recommended for large and medium spillways and large canal structures. The length of basin is 33% reduced with the use of appurtenances. The basin contains chute blocks, dentated sill. No baffle piers are used because high velocities might cause cavitation on piers (i.e. velocity > 15 m/sec). See figure (3.11), (Varshney).

b.4. USBR Stilling Basin III: It is used when the incoming velocities not exceed 15, atypical design of USBR stilling basin III is shown in figure (3.9).

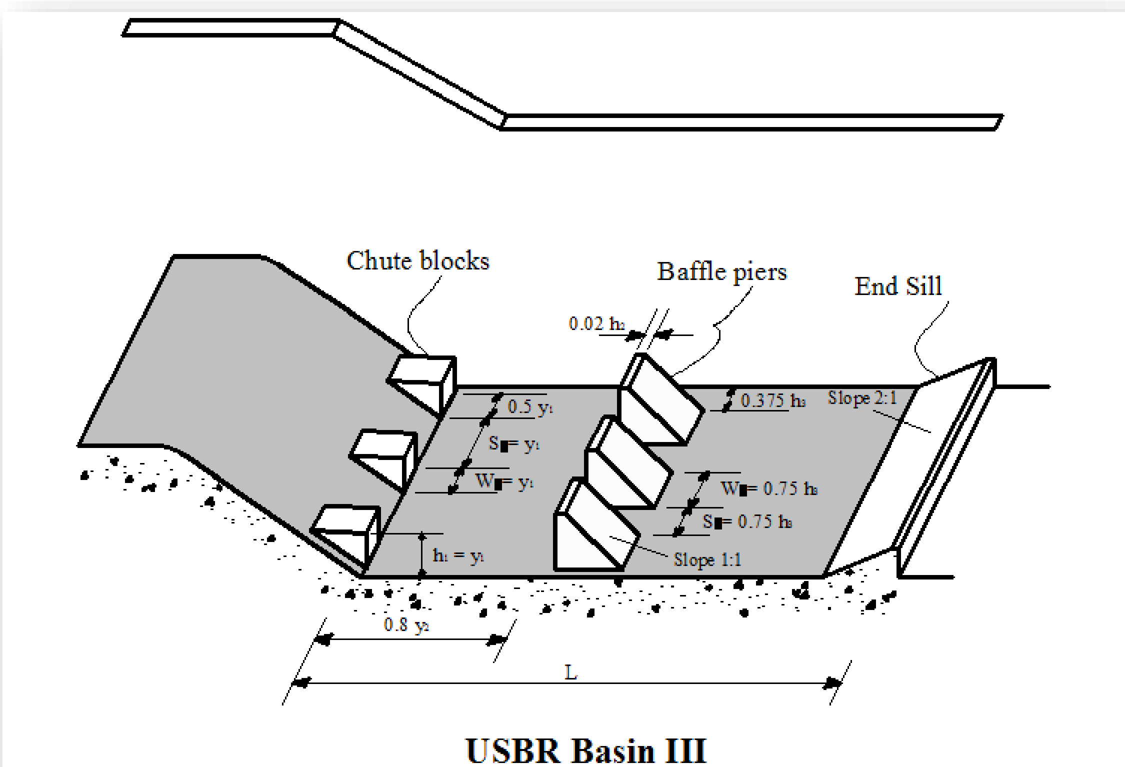


Figure (3.9): Typical Sketch for Stilling Basin III

3.12. Design Steps

1- Set apron elevation to use tailwater depth (T_w) plus an added factor of safety (use Fig. (3.11b), Chow) on the figure there is a minimum T_w depth line which indicates the point at which the front of the jump moves away from the chute blocks. Any lowering would cause the jump to leave the basin. Bureau recommends a minimum safety margin of 5% of D_2 be added to sequent depth.

2- Basin II maybe effective down to Froude No. 4.

3- Length of the basin can be obtained from Fig. (3.11c), Chow.

4- Height of chute blocks= D_1, width and spacing= D_1. Space of $0.5D_1$ is preferable long each wall to reduce spray and maintain desirable pressure.

5- Height of the dentated sill= $0.2D_2$. Maximum width and spacing= $0.15\ D_2$. The block is placed adjacent to each side wall. The slope of the continuous portion of the end sill is 2H: 1V.

6- The slope of the chute varies from 0.6 H: 1V to 2H: 1V.

7- This type of basin is suitable for spillways with fall up to 200 ft and flows 500 cfs/ft of basin width.

8- Water surface and pressure profiles of a jump in the basin are shown in Fig. (3.11d), Chow.

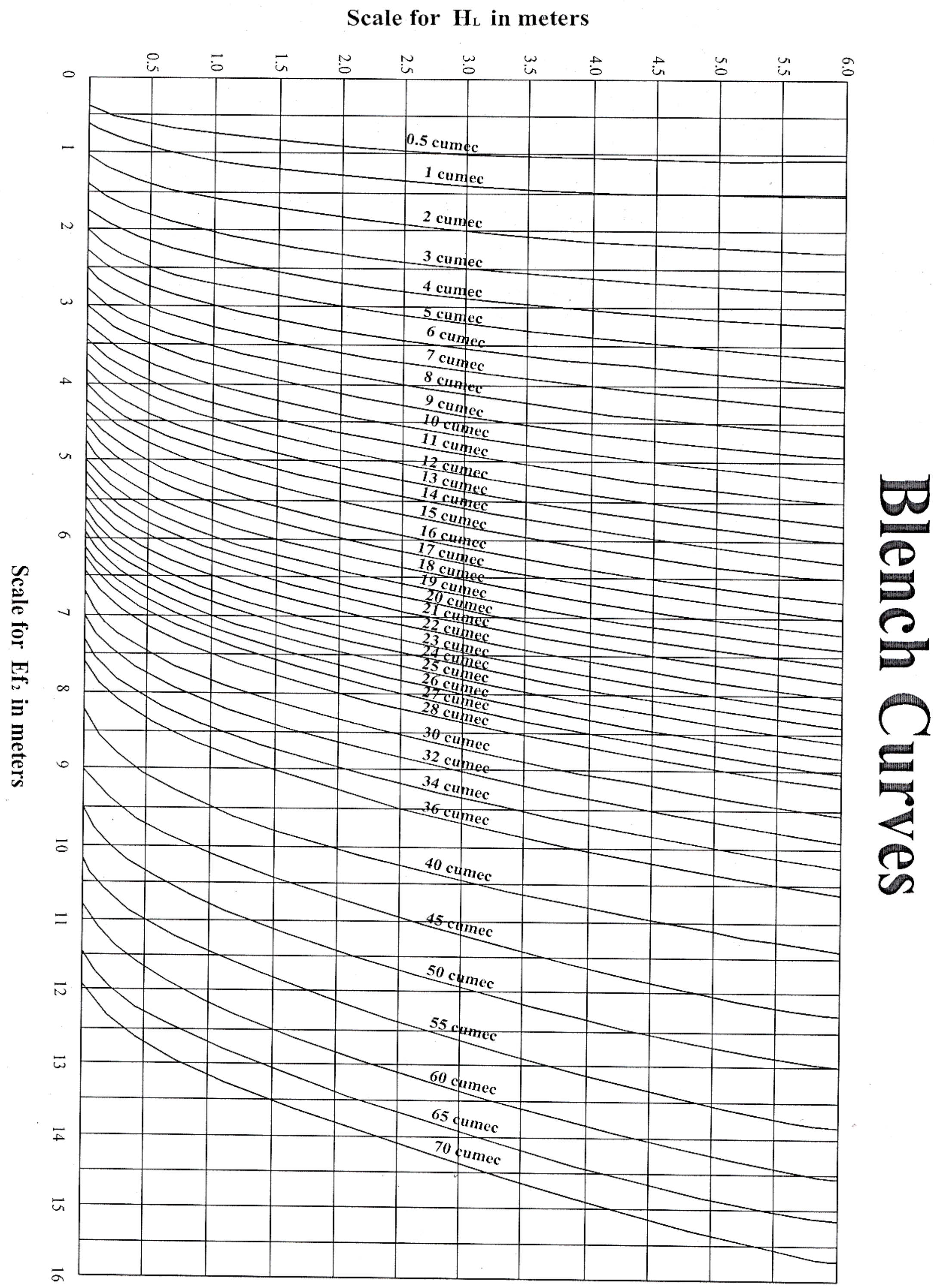


Figure (3.10): Blench Curves

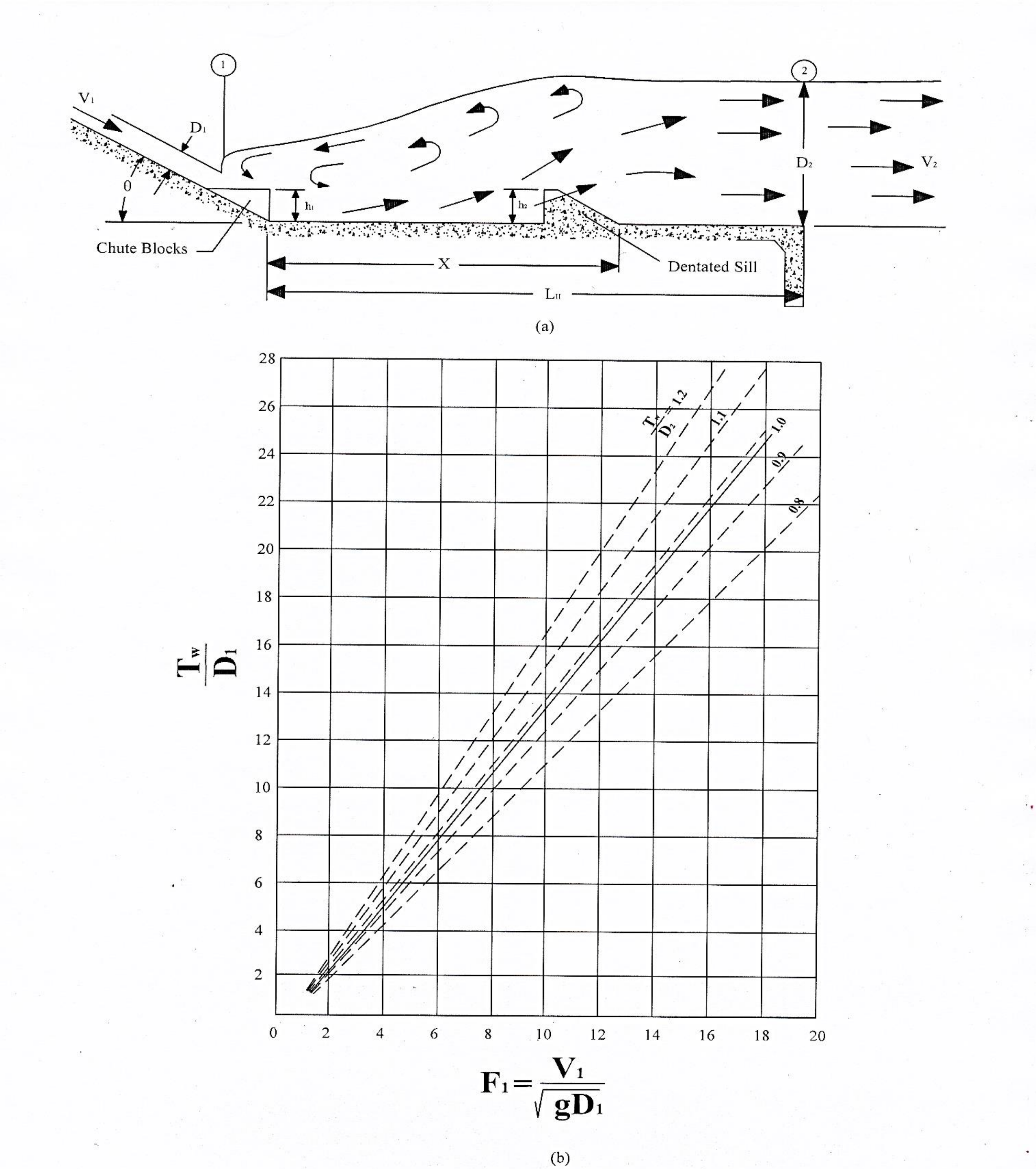


Figure (3.11): Continued

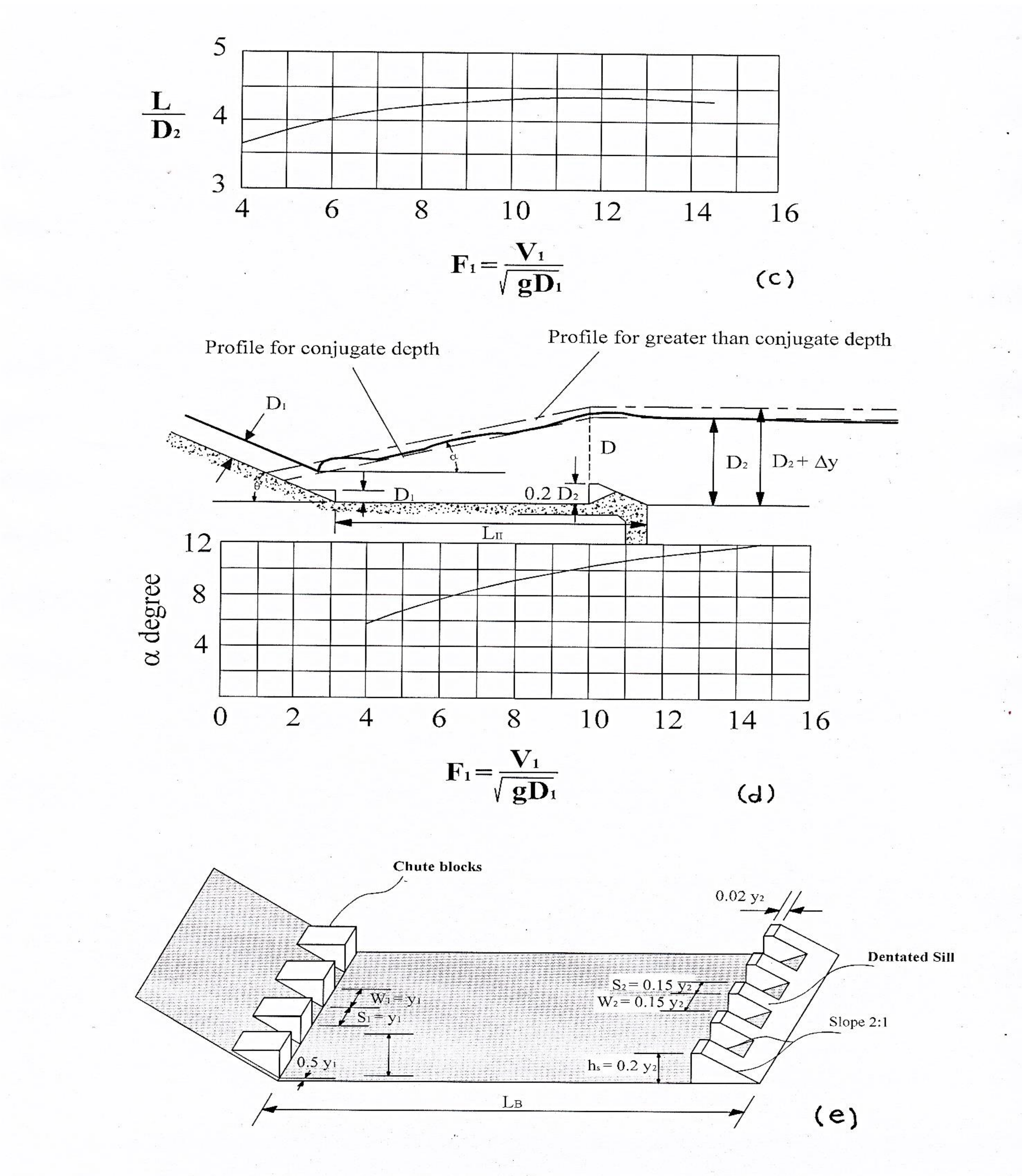


Figure (3.11): Design Curves and Proportion of USBR II (a) Definition of Symbols (b) Minimum Tail Water Depth (c) Length of Hydraulic Jump (d) Approximate Water Surface Profile and Pressure Profiles (Conjugate Depth = Sequent Depth) (e) Recommended Proportions

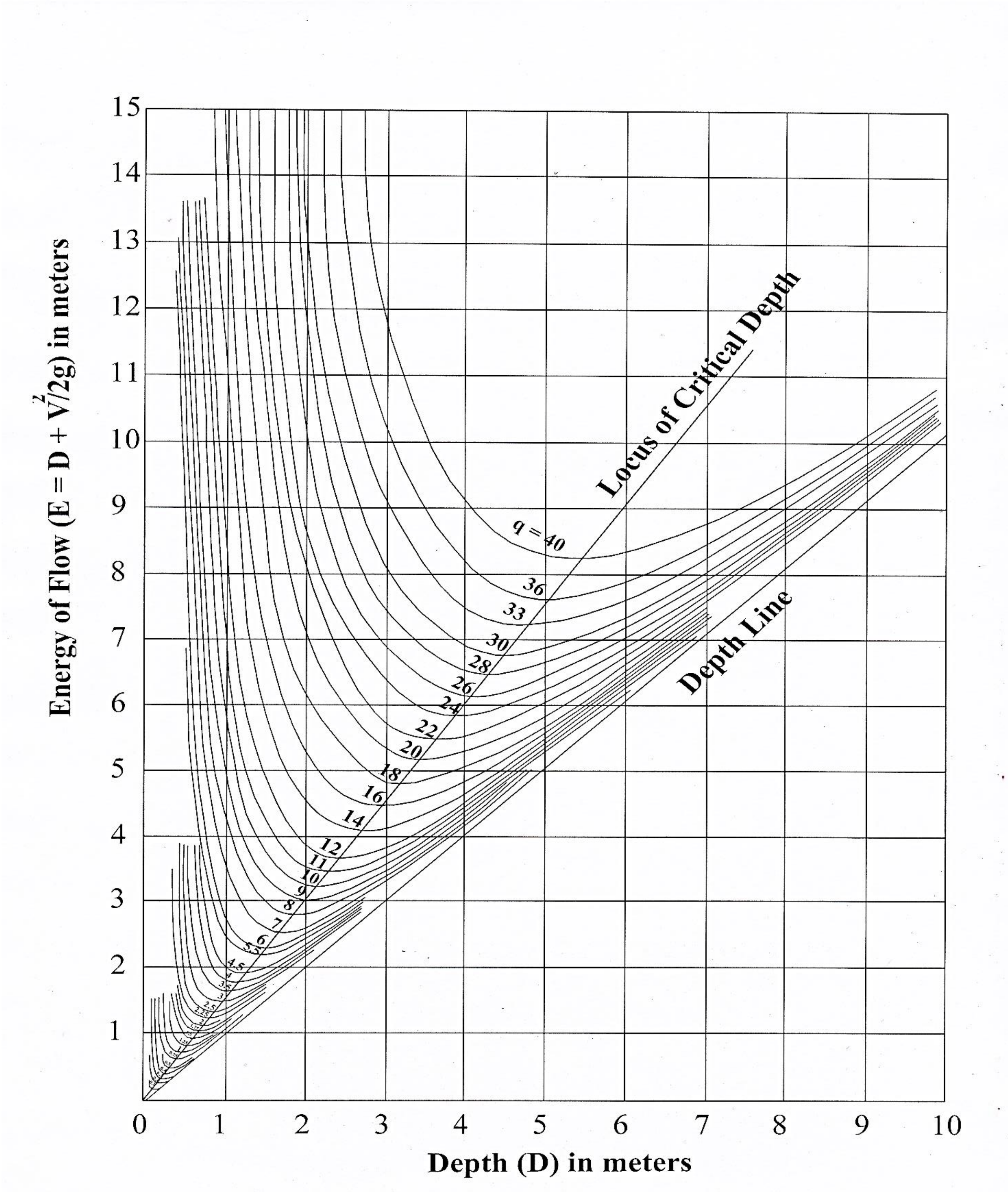


Figure (3.12): Energy Curves

Example 3.6: R.S. Varshney Stilling Basin

Water flows under a sluice gate discharges into a rectangular plain stilling basin. Determine the stilling basin parameters for the following data.

q =34 m^3 / s/ m, HL = 1 m, tail water depth = 8.3 m, width of basin = 20 m

Solution:

Design

From figure (3.10)(blench curves), $E_{f\,2} = 8.5$ m

$E_{f\,1} = E_{f\,2} + H_L = 8.5+1 = 9.5$ m

From energy of flow curves (figure (3.12)):

$D_1 = 2.9$ m , $D_2 = 7.3$ m , $D_3 = T_w = 8.3$ m

$V_1 = q / D_1 = 34/2.9 = 11.72$ m/s

$$Fr_1 = \frac{V_1}{\sqrt{gD_1}} = \frac{11.72}{\sqrt{9.81*2.9}} = 2.2 < 4.5 \quad \text{O.K}$$

$$D_c = \sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{34^2}{9.81}} = 4.9 \text{ m}$$

Length of Cistern

$L = 6\,(D_2 - D_1) = 6\,(7.3 - 2.9) = 26.4$ m

From F_r and L/D_1 relation

$L/D_1 = 12$, $L = 12*2.9 = 34.8$ m

$L = (26.4+34.8)/2 = 30.6 \approx 31$ m

Blocks

a. Chute blocks

1. Height h_c and F_r relation

   $h_c = 0.9$ m

2. Width and spacing = $0.75\, h_c = 0.75 * 0.9 = 0.675 \approx 0.7$ m

b. Floor blocks

1. Position

From $\frac{L}{D_3}$ *and* $\frac{L_B}{D_3}$ relation

$L_B = 0.9 * 8.3 = 7.5$ m

From L and L_B relation, $L_B = 6.7$ m

Average $L_B = \frac{6.7+7.5}{2} = 7.1\ m$

2. Height, h_B

From $\frac{D_2}{D_3}$ *and* $\frac{h_B}{D_3}$ relation

$\frac{D_2}{D_3} = \frac{7.3}{4.9} = 1.49$, $\frac{h_B}{D_3} = 0.22$, $h_B = 0.22 * 8.3 = 1.83$ m

From $\frac{D_2}{D_c}$ *and* $\frac{h_B}{D_c}$ relation

$\frac{D_2}{D_c} = \frac{7.3}{4.9} = 1.49$, $\frac{h_B}{D_c} = 0.31$, $h_B = 0.31 * 4.9 = 1.52$ m

Adopt mean value = $\frac{1.83+1.52}{2} = 1.67\ say\ 1.7\ m$

3. Width and spacing W_B

$\frac{W_B}{h_B} = 0.8$, $W_B = 0.8 * 1.7 = 1.36$ say 1.4 m

Top width = 0.2 h_B = 0.2 * 1.7 = 0.34 say 0.4 m

c. End sill

1. Height, He (relation between Fr and $\frac{H_e}{D_3}$)

$\frac{H_e}{D_3} = 0.18$, $H_e = 0.18 * 8.3 = 1.5$ m

From $\frac{H_e}{D_c}$ *and* $\frac{D_3}{D_c}$ relation

$\frac{D_3}{D_c} = \frac{8.3}{4.9} = 1.7$, $\frac{H_e}{D_c} = 0.3$, $H_e = 1.47$

Average $H_e = \frac{1.5+1.47}{2} = 1.485$ say 1.5 m

2. Top width = 0.02 D_2 = 0.02 * 7.3 = 0.146 say 0.15 m
3. Width and spacing of dents = 0.15 D_2 = 1.1 m
4. Width of basin = 20 m

H.W. 1: Find out stilling basin parameters (R. S. Varshney) for the following flow data for under sluice bays of a barrage across an alluvial.
Discharge intensity 28 $m^3/sec/m$
Head loss H_L = 0.8 m, Prejump depth D_1 = 2.68 m, Conjugate depth D_2 = 6.1 m
Tailwater depth T_w = 7.32 m

3.13. Hydraulic Design of Canal falls

3.13.1. Vertical Drop Fall

The energy is dissipated by means of impact and deflection of velocity suddenly from the vertical to the horizontal direction.

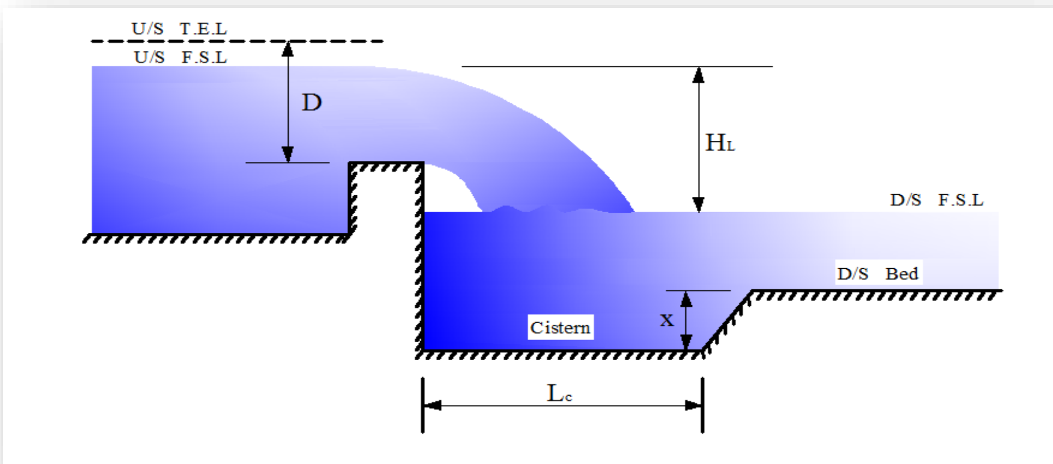


Figure (3.13): Cistern Element of Vertical Drop Fall

Cistern length (L_C) = $5\sqrt{H_L * D}$

Cistern depth (X) = $\frac{1}{4}\sqrt[3]{(H_L * D)^2} = \frac{d_c}{3}$

$d_c = (\frac{q^2}{g})^{1/3}$

L_c = the length of cistern

X = the depression below downstream bed

H_L = drop in meter

D = depth of crest below u/s T.E.L in meter

3.13.2. Design of Sarda Type Fall

1. Crest

a. Length of Crest (L_{cr}) = Bed width (BW)

Or : L_{cr} = BW + depth

b. Shape of Crest :

b.1. If $Q < 15\ m^3/s$:

- the section is rectangular and D/S face is vertical
- Top width (TW) = $0.55\sqrt{D_1}$
- Min. base width ($BW_{min.}$) = $D_1/2$

Where D_1 is the height of crest above downstream bed level. It may be capped with 25 cm – 1:2:4 cement concrete with its both edges rounded.

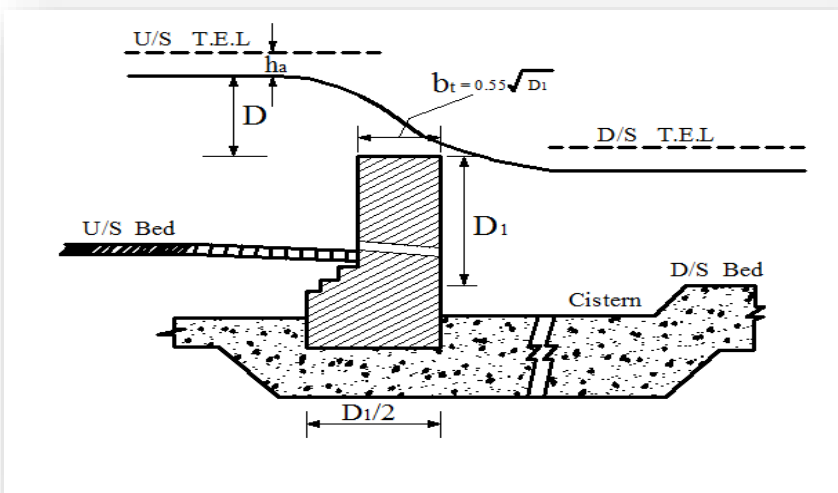


Figure (3.14): Rectangular Crest – Sarda Type Fall

b.2. If $Q > 15\ m^3/s$:

- the section is trapezoidal
- Top width (TW) = $0.55\sqrt{D + D_1}$
- U/S side slopes of 1:3
- Segment top conforming as a quadrant of a circle of 0.3 m in radius at D/S side
- D/S side slope = 1:8

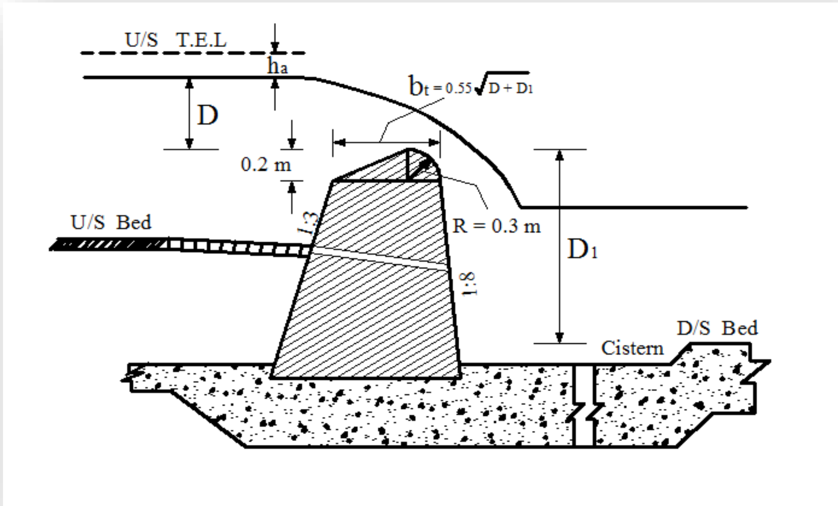


Figure (3.15): Trapezoidal Crest – Sarda Type Fall

c. Level of Crest :

$$Q = C\ L_t\ D^{1.5}\left(\frac{D}{B_t}\right)^{1/6}$$

L_t = length of crest

B_t = width of crest

The value of C for rectangular crest 1.835 and for trapezoidal crest 2.26

Crest level = U/S F.S.L. + h_a – D

U/S Curtain Wall

Curtain wall thickness = 1.5 * brick thickness

Depth of curtain wall = $\frac{1}{3}$ * water depth + 0.6 $\geq$ 0.8 m

2. Impervious Concrete Floor

a. Total length and its disposition

The minimum length of the floor on the D/S side is given as

$L_b = 10.53\ d_c + 4.877 - 1.5\ H_L$

L_b = downstream floor length.

This equation is used for clear falls and submergence less than 33%.

The balance of the total length may be provided under and U/S of crest.

b. Floor Thickness

The minimum U/S floor thickness is 0.3 m. The D/S thickness should be determined by uplift pressure with minimum of 0.6 m for large works (Khosla theory) and 0.3 m for minor works (Bligh theory).

4. Cistern

a. Length of cistern = $3.8d_c + 0.415 + H_L$

b. depth of cistern (X) = $\frac{d_c}{3}$ **in all cases**

$X = \frac{1}{4}\sqrt[3]{(H_L * D)^2}$

Example 3.7: Design a Sharda type fall with the data given below:
Full supply flow rate U/S / D/S = 10 cumecs
Drop = 1m
Full supply level U/S / D/S =101.5/100.5 m
Full supply depth U/S / D/S = 1.5/1.5 m
Bed level U/S / D/S = 100/99 m
Bed width U/S / D/S = 8m/8 m
Side slope = 1H: 1V
Soil is good loam and assume Bligh's coefficient = 7
Solution

1. Length of crest

Take crest length = L_t = 8m

2. Crest level

Since discharge less than 15 cumecs, rectangular crest with both sides vertical.

$$Q = CL_t D^{3/2}\left(\frac{D}{B_t}\right)^{1/6}$$

Assume $\mathbf{B_t = 0.8}$ **m. assumed value range (0.75-1.0) m**

$$10 = 1.835 * 8 * D^{3/2}\left(\frac{D^{1/6}}{0.8^{1/6}}\right)$$

$0.6545 = D^{1.67}$

D = 0.776 m say 0.78 m.

Velocity approach with 1:1 sides V_a=Q/A = $\frac{10}{(8+1.5*1)*1.5}$ = 0.702 m/sec

Velocity head=h_a= $\frac{v_a^2}{2g}=\frac{0.702^2}{2*9.81}=0.025\,\text{m}$

U/S T.E.L = U/S F.S.L + h_a = 101.5 + 0.025 = 101.525 m

R.L. of crest (U/S T.E.L – D) = 101.525 – 0.78 = 100.745 m Say 100.75m

Adopt crest level = 100.75 m

3. Shape of crest

i. Top width:

$B_t=0.55\sqrt{D_1}$,

D_1 = 100.75 – 99 = 1.75 m

∴ $B_t=0.55\sqrt{1.75}=0.73\text{m}$

Adopt B_t = 0.75 m

Check for D

$$10=1.835*8*D^{3/2}\left(\frac{D^{1/6}}{0.75^{1/6}}\right) \Longrightarrow D = 0.771 \text{ m}$$

ii. width and base = 0.5*D_1 = 0.5*1.75 = 0.875 m say 1 m

Its top shall be capped with 25 cm thick cement concrete.

4. Cistern Element

$$d_c=\left(\frac{q^2}{g}\right)^{1/3}=\left(\frac{(10/8)^2}{9.81}\right)^{1/3}=0.542\ m$$

Depth of cistern = $\frac{d_c}{3}=\frac{0.542}{3}=0.181\text{m}$

Cistern depth (X) = $\frac{1}{4}(H_L*D)^{2/3}=\frac{1}{4}(1*0.75)^{2/3}=0.21\text{m}$

Length of cistern (L_c) = 3.8d_c + 0.415 + H_L = 3.8*0.542 + 0.415 + 1 = 3.47 m

L_c = 5(H_L*D)$^{1/2}$ = 5*(1*0.75)$^{1/2}$ = 4.3 m

Provide 4.5 m long cistern at R.L. 98.75 m

5. Length of impervious floor

Bligh's coefficient =C= 7

Maximum static head =H=(crest level – d/s bed level)= 100.75 – 99 = 1.75 m

Total floor length required =C*H= 7*1.75 = 12.25 m

Minimum D/S floor length (L_p) required

$L_b = 10.53 d_c + 4.877 - 1.5 H_L = 10.53 * 0.542 + 4.877 - 1.5 * 1$ = 9.08 m say 9 m

6. Floor thickness

Minimum floor thickness of 0.3 m should be provided at the U/S region.

Max Up left head at the toe of crest = $\frac{1.75}{12.25} * (12.25 - 3.25) = 1.29\,\text{m}$

Floor thickness required = $\frac{1.29}{1.25} = 1.03\,\text{m}$

Provide 1.05 m thick concrete over laid with 0.2 m thick brick pitching.

Max up left head at 2.25 m D/S from the toe of crest $= \frac{1.75}{12.25} * (12.25 - 5.5) = 0.96 \; m$

Floor thickness required = $\frac{0.96}{1.25} = 0.77\,\text{m}$

Provide 0.8 m thick concrete over laid with 0.2 m thick brick pitching.

Floor thickness required at 4.5 m D/S from the toe of crest $= \frac{1.75}{12.25} * \left(\frac{12.25 - 7.75}{1.25} \right) = 0.51 \; m$

Provide 0.55 m thick concrete over laid with 0.2 m thick brick pitching.

Floor thickness required at 6.75 m D/S from the toe of crest $= \frac{1.75}{12.25} * \left(\frac{12.25 - 10}{1.25} \right) = 0.26 \; m$

Provide 0.3 m thick concrete over laid with 0.2 m thick brick pitching.

7. Curtain Walls

a. D/S Curtain Wall

The curtain walls at D/S end of floor should be 1.5 brick thick and of depth $\left(\frac{d}{2} + 0.6m \right)$ to a minimum of 1 m.

Depth of curtain wall at D/S end floor = $\frac{1.5}{2} + 0.6 = 1.35$ m (Provide 0.4 m*1.4 m deep curtain wall).

b. U/S Curtain Wall

Depth = $\frac{u.s \; water \; depth}{3} + 0.6$ = 0.5 + 0.6 = 1.1 m

Provide 0.4 m*1.1 m deep curtain wall.

Example: Design a Sarda type fall for data given below:

Full supply flow rate U/S / D/S = 50 cumecs

Drop = 1.5 m

Full supply level U/S / D/S = 101.5/100 m

Full supply depth U/S / D/S = 2 m /2 m

Bed level U/S / D/S = 99.5 m/98 m

Bed width U/S / D/S = 30 m/30 m

Side slope = 1H: 1V

Safe exit gradient = 1/4.5

Design the floor using Khosla's theory.

Solution

1. Length of crest

Take crest length (L_t) = 30 m

2. Crest level

Since discharge greater than 15 cumecs, use trapezoidal crest with C = 2.26

$$Q = CL_t D^{3/2}\left(\frac{D}{B_t}\right)^{1/6}$$

Assume B_t = 1 m.

$$50 = 2.26 * 30 * D^{3/2}\left(\frac{D^{1/6}}{1^{1/6}}\right)$$

$D^{1.67}$ = 0.7375, D = 0.833 m.

Velocity approach with 1:1 sides $V_a = \frac{50}{30*2+2*2} = 0.781$ m/sec

Velocity head=$h_a = \frac{v_a^2}{2g} = \frac{0.781^2}{2*9.81} = 0.031\text{m}$

U/S T.E.L = U/S F.S.L + h_a = 101.5 + 0.031 = 101.531 m

R.L. of crest = U/S T.E.L – D = 101.531 – 0.833 = 100.698 m

3. Shape of crest

i. Top width:

$B_t = 0.55\sqrt{D + D_1}$,

D_1 = crest level - D/S bed level

D_1= 100.698 – 98 = 2.698 m $= 0.55\sqrt{0.833 + 2.698} = 1.034$ m

Adopt B_t = 1.0 m, U/S face slope 1H: 3V and D/S face slope 1H: 8V.

4. Curtain Walls

a. D/S Curtain Wall

Depth of curtain wall $= \left(\frac{d}{2} + 0.6m\right) = \frac{2}{2} + 0.6 =$ 1.6 m

Adopt D/S curtain wall of 0.4m*1.6 m deep.

b. U/S Curtain Wall

Depth of curtain wall = $\frac{u.s\ water\ depth}{3} + 0.6 \ = \frac{2}{3} + 0.6 = 1.27$ m

Adopt U/S curtain wall of 0.4 m*1.3 m deep including 0.3 m foundation concrete.

5. Cistern Element

$$d_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{(50/30)^2}{9.81}\right)^{1/3} = 0.65\ m$$

X = $\frac{d_c}{3} = \frac{0.65}{3} = 0.22$m

X = $\frac{1}{4}(H_L * D)^{2/3} \ = \frac{1}{4}(1.5 * 0.833)^{2/3} = 0.29$ m (Adopt x = 0.3 m)

Cistern level 98 – 0.3 = 97.7 m

Length of cistern (L_c) = $3.8d_c$ + 0.415 + H_L = 3.8*0.66 + 0.415 + 1.5 = 4.423 m

$L_c = 5(H_L*D)^{1/2} = 5*(1.5*0.833)^{1/2}$ = 5.589 m (Adopt cistern length = 5.6 m)

6. Floor thickness and exit gradient

$$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$$

Max static head (H) =crest level – D/S bed level

H=100.698 – 98 = 2.698 m

G_E = 1/4.5 $\Longrightarrow$ $\frac{1}{4.5} = \frac{2.698}{1.7} * \frac{1}{3.14\sqrt{\lambda}}$ $\Longrightarrow$ $\lambda = 5.167$

$\alpha = [(2\lambda - 1)^2 - 1]^{1/2} = 9.282$

Total floor length required (b) = α*d = 9.282*1.7 = 15.77 m

Adopt floor length = 16 m

$L_b = 10.53 d_c + 4.877 - 1.5 H_L = = 10.53 * 0.66 + 4.877 - 1.5 * 1.5 =$ 9.577 m

Adopt D/S floor length of 10 m and the rest under and U/S of crest.

7. Pressure Calculation

Assume minimum U/S and D/S floor thickness near cut off = 0.6 m

a. U/S Curtain wall

b = 16 m, d = 1.3 m, α=b/d = 12.308, λ=6.674

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{1.3}{16} = 0.0815$$

$\phi_{D1} = 100 - \phi_D = 100 - 18 = 82\%$

$\phi_{C1} = 100 - \phi_E = 100 - 26 = 74\%$

ϕ_C correction due to floor thickness = $\frac{0.6}{1.3}(82 - 74) = +3.7\%$

ϕ_C corrected = 74 + 3.7 = 77.7%

b. D/S Curtain wall

d = 1.7 m, b = 16 m, α=b/d = 9.412, λ= 5.232 , $\frac{1}{\alpha} = \frac{d}{b} = \frac{1.7}{16} = 0.10625$

$\phi_E = 28\%$, $\phi_D = 20\%$

ϕ_E correction for thickness = $\frac{0.6}{1.7}(28 - 20) = -2.82\%$

ϕ_E corrected = 28 – 2.82 = 25.18%

8. Floor thickness

i. U/S Floor

The up lift pressure in the U/S is counter balanced by weight of water itself and therefore no thickness is required. However, provide 0.6 m thick concrete.

ii. D/S Floor

a. At toe of crest

i.e at distance 6m from U/S end.

H = crest level – D/S bed level = 100.698 – 98 = 2.7 m

% pressure = $25.18+\frac{52.52}{15.2}*9.6=58.35\%$

Up lift head due to static head = 0.5835* (2.7) = 1.575 m of water

Floor thickness required = $\frac{1.575}{1.25}=1.26$ m of concrete

Provide 1.3 m thick concrete floor at the toe of crest in a length of 2 m over laid 0.2 m thick brick pitching.

b. At 2 m from the toe of crest

% pressure = $25.18+\frac{52.52}{15.2}*7.6=51.44\%$

Uplift head due to static head = 0.5144* (2.7) = 1.39 m of water

Floor thickness required = $\frac{1.39}{1.25}=1.11$ m of concrete

Provide 1.2 m thick concrete floor over laid 0.2 m thick brick pitching.

c. At 4 m D/S from the toe of crest

% pressure = $25.18+\frac{52.52}{15.2}*5.6=44.53\%$

Up lift head due to static head = 0.4453* (2.7) = 1.2 m of water

Floor thickness required = $\frac{1.20}{1.25}=0.96$ m of concrete

Provide 1.0 m thick concrete floor over laid 0.2 m thick brick pitching.

d. At 6 m D/S from the toe of crest

% pressure = $25.18+\frac{52.52}{15.2}*3.6=37.62\%$

Up lift head due to static head = 0.3762* (2.7) = 1.01 m of water

Floor thickness required = $\frac{1.01}{1.25}=0.81$ m of concrete

Provide 0.9 m thick concrete floor over laid 0.2 m thick brick pitching.

e. At 8 m D/S from the toe of crest

% pressure = $25.18+\frac{52.52}{15.2}*1.6=30.71\%$

Up lift head due to static head = 0.3071*(2.7) = 0.83 m of water

Floor thickness required = $\frac{0.83}{1.25}=0.66$ m of concrete

Provide 0.7 m thick concrete floor over laid 0.2 m thick brick pitching.

Chapter Four

Hydraulic Design of Regulators

4.1. Introduction

A canal obtains its share of water from the pool behind a barrage through a structure called the ***canal head regulator*** (Figure 1). Though this is also a regulation structure for controlling the amount of water passing into the canal (with the help of adjustable gates), it shall be discussed under diversion works. In this lesson, attention is focused on structures that regulate the discharge and maintain the water levels within a canal network.

Figure (4.1): Neel Schaffer Regulator

4.2. Head and Cross Regulators

The supplies passing down the parent canal and off take channel are controlled by cross regulator and head regulator respectively.

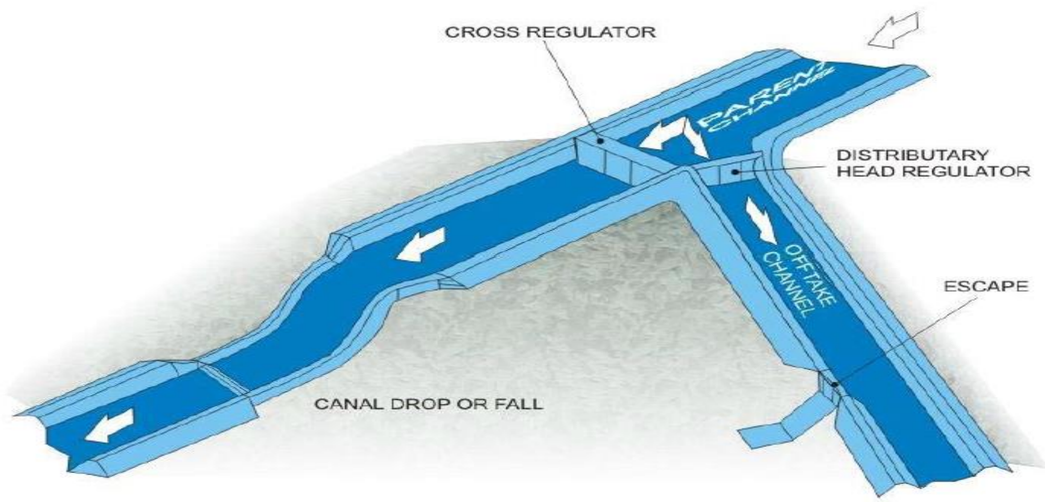


Figure (4.2): Cross and Head Regulator

4.2.1. Cross Regulator

A structure constructed across any canal (a main canal, a branch canal). Its regulators the quantity of water supplied and also the level of water on the u/s & D/S side.

4.2.1.1. Functions of Cross Regulators

1. Regulation of the canal system.

2. Raising the water level in the main canal in order to feed the off take channels.

3. To facilitate communication by building a road over the cross regulator with little extra cost.

4. To absorb the fluctuations in the canal system.

4.2.2. Head Regulator

A structure constructed at the head of an off-take channel.

4.2.2.1. Functions of Head Regulators

1. To regulate and control supplies entering the off take channel (distributary) from the main (parent) canal.
2. To control silt entering into the distributary.
3. To serve for measurement of discharge.
4. For shutting out the river floods.

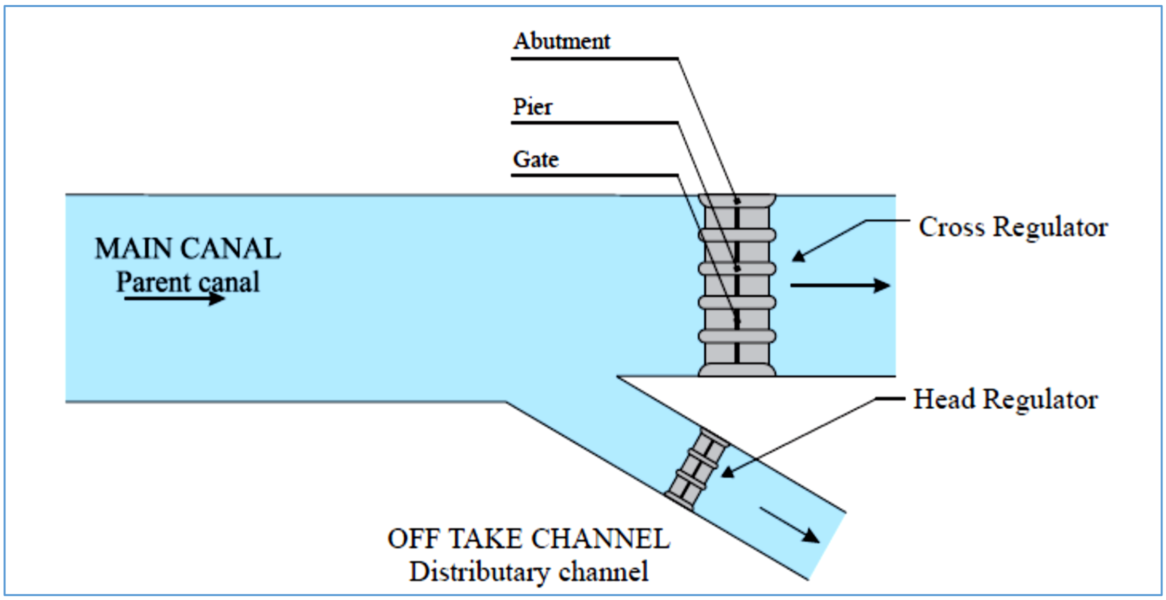


Figure (4.3): Layout of a Main Canal with Details of the Cross

4.3. Design of Cross & Head Regulator of Off-Take Channel (Design Criteria)

1. Waterway

The effective waterway of head regulator should not be less than 60% of bed width of off taking channel and mean velocity of flow for fully open regulator should not exceed 2.5 m/sec.

2. Crest level

Crest level of the distributary head regulator is generally kept 0.3 m to 0.6 m higher than crest level of cross regulator. The crest level of C.R. is provided at bed level of parent canal.

The head of water over the crest (H_e) should be worked out from the formula

$$Q = C\, B_e\, H_e^{3/2}$$

C = Coefficient of discharge

B_e = Effective length of crest = $B_t - 2(N\times K_p + K_a)H_e$

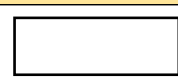
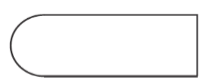
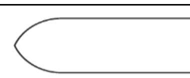
B_t = Net length of the crest

N = Number of piers

K_p = Pier contraction coefficient

K_a = Abutment contraction coefficient

Table (4.1): Coefficients of contraction for piers and abutments.

Type of pier	**Description**	**K_p**
Square nosed pier		0.02
Round nosed pier		0.01
Pointed nosed pier		0.005
Type of abutment		**K_a**
Square abutment		0.2
Round abutment		0.1

3. Coefficient of discharge (C)

The coefficient of discharge C is 1.84 for crests of width less than or equal to $\frac{2}{3}\,H_e$. In case of submerged falls, C should be reduced depending on the drowning ratio, see Fig. 4.4.

C =1.840 (H.R. crest)

C =1.705 (C.R. crest)

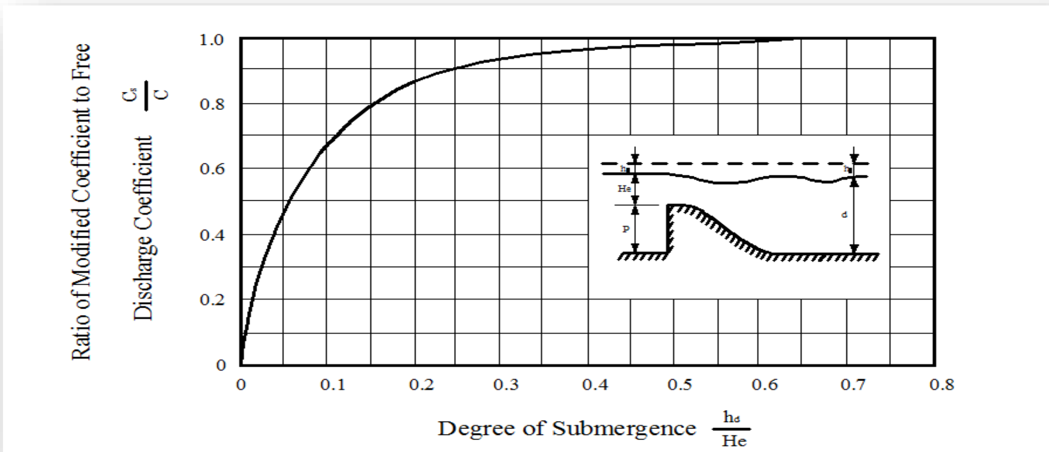


Figure (4.4): The Relation between C_s/C and h_d/H_e

4. Shape of crest

The U/S face of the crest should be given a slope of 1:1. The D/S sloping glacis should not be steeper than 2:1. The corners at the crest should be rounded as per details given in fig. 4.5.

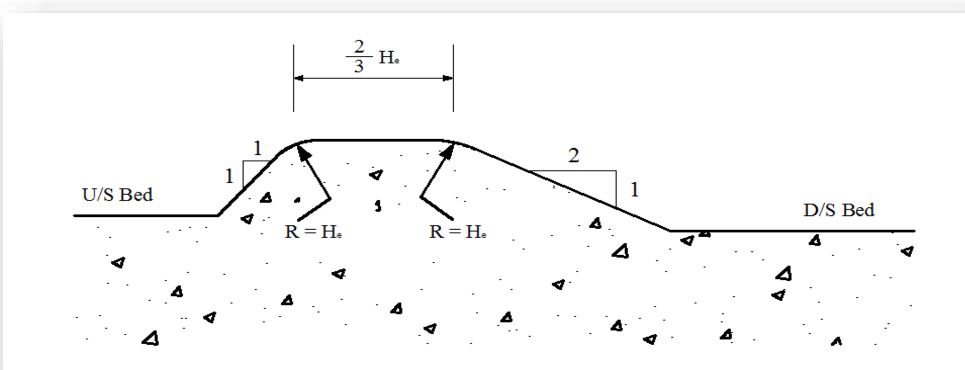


Figure (4.5): Sharp Crest Geometry

5. Crest width

For head regulator Crest width should be kept equal to $\frac{2}{3}\ H_e$

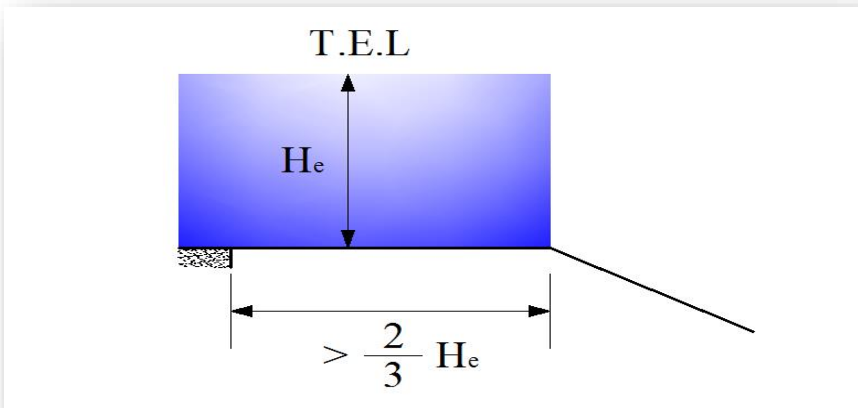


Figure (4.6): Crest Width

6. Level and Length of D/S Floor

Find discharge per meter length (q) and energy of flow on D/S (E_{f2}). (From Blench Curves, Fig. 3.10)

D/S floor level = water level at D/S (F.S.L. $_{D/S}$) - E_{f2}

If the actual bed level of canal at D/S lower than calculated value, design D/S floor level on it.

Find E_{f1}, D_1, D_2 (from fig. 3.12 – energy of flow curves)

Length of D/S floor = 5 ($D_2 – D_1$)

7. Vertical Cut-offs

The cut-off should be provided at the end of u/s. and d/s. floors for safety against uplift, scour, undermining and exit gradient. Due to Lacey's scour depth.

Table (4.2): Minimum Depth of Cut-off According to Discharge Values

Canal capacity(Q) (m^3/s)	Min. depth of U/S cut-off below bed level or G.L. whichever is lower (m)	Min. depth of D/S cut-off below bed level or G.L. whichever is lower (m)
Up to 3	1.0	1.0
3.1 - 30	1.2	1.2
30.1 - 150	1.5	1.5
Above 150	1.8	1.8

8. Thickness of Top Coat

Table (4.3): Thickness of Top Coat According to Discharge Values

Canal capacity (m^3/s)	**Thickness of top coat (*mm*)**
$Q \leq 1.5$	100
$1.5 \leq Q \leq 30$	150
$30 \leq Q \leq 150$	200
$Q > 150$	300

9. Freeboard

Table (4.4): Values of Freeboard According to Discharge Values

Canal capacity (m^3/s)	**Freeboard (*m*)**
$Q \leq 1.0$	0.3
$1 < Q \leq 10$	0.4
$10 < Q \leq 30$	0.6
$30 < Q \leq 150$	0.8
$Q > 150$	1.0

10. Protection Works

Downstream of floor, properly designed filter loaded by concrete blocks should be provided. The length of inverted filter is kept equal to 2D (D is the depth of D/S cut-off below D/S bed).

The width of gabs between the blocks shall not be more than 50 mm which should be packed with biggest size of pebbles available. Beyond the filter, an apron of 1.5D length shall be provided. Similar protection is also provided in the U/S in a length equal to D. The cubic content of material in launching apron should be equal to 2.25D m^3·m/m length.

Example 4.1: Design a cross regulator and a suitable head for a distributary which takes off at an angle of 60º from a canal which discharges 120 m^3/s

Discharge of distributary = 10 m^3/s

Bed width of distributary = 10 m

Water depth of distributary = 1.2 m

Full supply level of distributary = El. 310.2 m

Full supply level of parent channel (F.S.L.) $= \frac{\text{u.s.}}{\text{d.s.}} = \frac{311.00\ \text{m}}{310.85\ \text{m}}$

Bed width of parent channel $= \frac{\text{u.s.}}{\text{d.s.}} = \frac{60.0\ \text{m}}{56.0\ \text{m}}$

Depth of water in parent channel $= \frac{\text{u.s.}}{\text{d.s.}} = \frac{2.0\ \text{m}}{2.0\ \text{m}}$

Safe exit gradient (G.E.) $= \frac{1}{5}$

Solution

A. Design of Cross Regulator

1. Fixation of crest level and waterway of cross regulator

The crest level of the cross regulator will be provided at U/S bed level of the channel.

∴ Crest level of C.R. = F.S.L. of parent channel - water depth

= 311 - 2 = 309 m

The water way has been worked out on the basis of the following formula.

$$Q = C\, B_e\, H_e^{3/2}$$

Where C is to be worked out with the help of fig. 4.4 depending on the degree of submergence.

Degree of submergence $= \frac{h_d}{H_e}$

h_d = T.E.L.$_{U/S}$ - F.S.L.$_{D/S}$ = 311 - 310.85 = 0.15 m

H_e = F.S.L.$_{U/S}$ - crest level = 311 – 309 = 2 m

$$\therefore \frac{h_d}{H_e} = \frac{0.15}{2} = 0.075$$

From Fig. 4.4

$$\frac{C_s}{C} = 0.56 \quad , \quad \text{where } C = 1.705$$

∴ C_s = 1.705 * 0.56 = 0.95

Substituting the values in above formula

$$120 = 0.95 * B_e * 2^{3/2}$$

∴ B_e = 44.66 m say 45m

Assume 6 bays of 8 m each = 48 m

Use rounded nose piers & square abutment.

Find net length of crest

$B_t = B_e + 2(N*K_p + K_a)\ H_e$

$= 45 + 2\ (5*0.01 + 0.2) * 2$

$= 46 < 48$ O.K.

Provide 5 piers of width 1.6 *m* each.

Total waterway = (6*8) + (5*1.6) = 56 m O.K.

2. Level and length of downstream floor

$Q = 120\ m^3/s$

$$q = \frac{Q}{B_t} = \frac{120}{48} = 2.5\ m^3/sec/m$$

$H_L = T.E.L._{U/S} - T.E.L._{D/S} = F.S.L._{U/S} - F.S.L._{D/S} = 311 - 310.85 = 0.15$ m ($v^2/2g$ is negligible)

From Blench Curves, Fig. 2.5

$E_{f2} = 1.435$m

D/S floor level = $F.S.L._{D/S} - E_{f2} = 310.85 - 1.435 = 309.415$ m

Actual cistern level = 310.85 - 2 = 308.85 m lower than 309.415 m O.K.

Cistern length $L_c = 5\ (D_2 - D_1)$

$E_{f1} = E_{f2} + H_L = 1.435 + 0.15 = 1.585$ m

From energy of flow curves (Fig. 2.7)

$E_{f1} = 1.585$ m gives $D_1 = 0.534$ m

$E_{f2} = 1.435$ m gives $D_2 = 1.3$ m

$\therefore L_c = 5\ (1.3 - 0.534) = 3.83$ m say 4.0 m

3. Vertical cutoffs

Q =120 m^3/s

Referring to table 6.1, the minimum depth of U/S and D/S cutoff = 1.5 m.

U/S cutoff is at El. = 309 -1.5 = 307.5 m

4. Total floor length and exit gradient

The floor will be subjected to maximum uplift pressure when full supply level is maintained on the U/S for feeding distributary but no water is flowing down the cross regulator.

Exit gradient:

$$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$$

U/S full supply level = 311.00

D/S floor level = 308.85

∴ Max. static head (H) = $F.S.L._{U/S}$ - $G.L._{D/S}$ = 311.00 – 308.85 = 2.15 m

Find economical total floor length (b) by trial & error (length of D/S floor $\leq \frac{2}{3}b$)

Min. length of D/S floor = 4 + (3× (U/S bed level – D/S bed level)

= 4+ (3× (309-308.85)) = 4.45 m

$\frac{1}{5} = \frac{2.15}{1.5}\frac{1}{\pi\sqrt{\lambda}} \Rightarrow \lambda = 5.2;\ \lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \Rightarrow 5.2 = \frac{1+\sqrt{1+\alpha^2}}{2}$

∴α = 9.35

b = α d

b = 9.3×1.5 = 14.0 m

D	λ	α	b	2/3 b
1.5	5.2	9.35	14	9.33
1.6	4.57	8.08	12.93	8.62
1.7	4.05	7.03	11.95	7.96
2.0	2.92	4.75	9.5	6.33
2.25	2.31	3.48	7.84	5.22
2.5	1.87	2.56	6.39	4.26

D/S cutoff is at El. = 308.85 – 2.25 = 306.6 m

D/S floor length = 5.0 m

U/S. floor length = 3.0 m

Total floor length = 8.0 m

5. Pressure calculation

a. Upstream cutoff (d = 1.5 m, b = 8 m)

Assume U/S floor thickness near cut off =0.6 m ($Q > 1.5\ m^3/sec$)

ΦE_1= 100% of head

$$\alpha = \frac{b}{d} = \frac{8}{1.5} = 5.33$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+5.33^2}}{2} = 3.21$$

$$\Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{3.21-2}{3.21}\right) = 37\%$$

$$\Phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{3.21-1}{3.21}\right) = 26\%$$

$$\Phi_{C1} = 100\% - \Phi_E = 100\% - 37\% = 63\% \text{ of head}$$

$$\Phi_{D1} = 100\% - \Phi_D = 100\% - 26\% = 74\% \text{ of head}$$

Correction of Φ_{C1} for floor thickness $= +\frac{t}{d}(\Phi_{D1} - \Phi_{C1}) = \frac{0.6}{1.5}(74 - 63) = +4.4\%$

Correction of Φ_{C1} due to interference of D/S cutoff

$$C = \pm 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = +19\sqrt{\frac{1.8}{7}}\left(\frac{0.9+1.8}{8}\right) = +3.2\%$$

$\therefore \Phi_{C1}$ corrected = (63+4.4+3.2) % = 70.6% of head

b. Downstream cutoff

d=2.25 m, b=8 m

Assume D/S floor thickness near cut off = 0.6 m (Q > 1.5 m^3/sec)

$$\alpha = \frac{b}{d} = \frac{8}{2.25} = 3.55$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+3.55^2}}{2} = 2.34$$

$$\Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{2.34-2}{2.34}\right) = 46\%$$

$$\Phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{2.34-1}{2.34}\right) = 31\%$$

$$\Phi_C = 0\%$$

Correction of Φ_E for floor thickness $= -\frac{t}{d}(\Phi_E - \Phi_D) = -\frac{0.6}{2.25}(46 - 31) = -4.0\%$

Correction of Φ_E due to interference of U/S cutoff

$$C = \pm 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = -19\sqrt{\frac{0.75}{7}}\left(\frac{1.65+0.75}{8}\right) = -1.8\%$$

$\therefore \Phi_E$ corrected = (46-4-1.8) % = 40.2% of head

6. Floor thickness

a. D/S Floor

1. At 2 m From D/S End

$\%\text{pressure} = 40.2 + \left(\frac{70.6-40.2}{7}\right) \times 1.5 = 47\% \text{ of head}$

Max. static head (H) = 2.15 m

Head = H × % pressure = $2.15 \times \frac{47}{100} = 1.0 \text{ m of water}$

Minimum concrete thickness $t_{\min} = \frac{\text{Head}}{\gamma_c - \gamma_w} = \frac{1.0}{2.4-1} = 0.71 \text{ m}$

Provide 0.75 m thick concrete floor for 1.5 m.

2. At 4 m From D/S End

$\% \text{ pressure} = 40.2 + \left(\frac{70.6-40.2}{7}\right) \times 3.5 = 55.4\% \text{ of head}$

Head $=2.15 \times \frac{55.4}{100} = 1.19 \text{ m of water}$

$\therefore t_{\min} = \frac{\text{Head}}{\gamma_c - \gamma_w} = \frac{1.19}{2.4-1} = 0.85 \text{ m}$

Provide 0.9 m thick concrete floor for 2 m.

B. Design of Distributary Head Regulator

1. Fixation of Crest and Waterway

The crest level should be provided 0.5 m higher than upstream floor level.

Crest level = 309 + 0.5 = 309.5 m

h_d = T.E.L. $_{U/S}$ - T.E.L. $_{D/S}$ = 311 - 310.2 = 0.8 m

H_e = F.S.L. $_{U/S}$ - Crest level = 311 – 309.5 = 1.5 m

$\therefore \frac{h_d}{H_e} = \frac{0.8}{1.5} = 0.533$

From Fig. 4.4

$\frac{C_s}{C} = 0.98$, where $C = 1.84$

$\therefore C_s = 1.84 \times 0.98 = 1.8$

The effective width of waterway is found by

$Q = C\, B_e\, H_e^{3/2} \rightarrow 10 = 1.8\, B_e\, 1.5^{3/2} \rightarrow \therefore B_e = 3\ m$

Provide 60% of distributary width = 0.6 *10 = 6 m

Provide 2 bays of 3 m each separated by 1 m thick pier.

Therefore, the overall waterway 2*3 + 1= 7 m

2. Level and Length of D/S Floor

Q = 10 m^3/s, water way = 6 m

$q = \frac{Q}{B_e} = \frac{10}{6} = 1.67\ m^3/sec/m$

Head loss H_L = 311 - 310.2 = 0.8 m

Using Blench curves (Fig. 3.5)

$H_L = T.E.L._{U/S} - T.E.L._{D/S} = F.S.L._{U/S} - F.S.L._{D/S}$ = 311 - 310.2 = 0.8 m ($v^2/2g$ is negligible)

Cistern length $L_c = 5\,(D_2 - D_1)$

$E_{f1} = E_{f2} + H_L$ =1.37 + 0.8 =2.17 m

From energy of flow curves (Fig. 2.7)

E_{f2} = 1.37 m gives D_2 = 1.32 m

E_{f1} = 2.17 m gives D_2 = 0.32 m

$\therefore L_c = 5\,(1.32 - 0.32) = 5.0$ m

Provide cistern length = 6m

D/S floor level = $F.S.L._{D/S} - E_{f2}$ = 310.2 - 1.37 = 308.83 m

Actual cistern level = 310.2 - 1.2 = 309.0 m > 308.83 m.

Provide D/S floor level at El. 308.8 (Ground Level)

3. Vertical cutoffs

a. U/S Cutoff

Q =10 m^3/s

Referring to table 6.1.

Provide U/S cutoff depth 1.5 m

U/S cutoff is at El. = 309 - 1.5 = 307.5 m

b. D/S Cutoff

The minimum D/S cutoff depth = 1.2 m

4. Total Floor Length and Exit Gradient

Exit gradient:

$$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$$

U/S full supply level = 311.00

D/S floor level = 308.85

∴ Max. static head (H) = F.S.L.$_{U/S}$ - G.L.$_{D/S}$ = 311.00 – 308.2 = 2.2 m

Find economical total floor length (b) by trial & error (length of d.s. floor $\le \frac{2}{3}b$)

D/S glacis length 2H:1V slope = 2 (309.5 – 308.8) = 1.4 m
Min. length of D/S floor = 5+ (2×(crest level – D/S bed level) =6 + (2×(309.5 - 308.8))

= 7.4 m

$\frac{1}{5} = \frac{2.2}{1.2}\frac{1}{\pi\sqrt{\lambda}} \Rightarrow \lambda = 8.50;\ \lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \Rightarrow 8.5 = \frac{1+\sqrt{1+\alpha^2}}{2}$

∴α = 15.99

b = α d

b = 15.99 * 1.2 = 19.0 m

d	λ	α	b	2/3 b
1.2	8.50	15.99	19.00	12.79
1.4	6.26	11.47	16.05	10.7
1.6	4.75	8.50	13.62	9.08
1.7	4.24	7.42	12.60	8.40> 7.4 OK

∴D/S cutoff is at El. = 308.8 – 1.7 = 307.1 m

H_e = F.S.L. $_{U/S}$ - crest level = 311 – 309.5 = 1.5m

Crest width $= \frac{2}{3} H_e = \frac{2}{3} \times (1.5) = 1.0\ m$

U/S glacis length 1H:1V slope = 1(309.5-309.0) = 0.5 m

U/S. floor length = 4.1 (balanced)

Total floor length = 13 m

5. Pressure calculations

a. Upstream Cutoff

Assume U/S floor thickness near cut off =0.6 m (Q > 1.5 m^3/sec)

ΦE_1= 100% of head

$$\alpha = \frac{b}{d} = \frac{13}{1.5} = 8.67$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+8.67^2}}{2} = 4.86$$

$$\Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{4.86-2}{4.86}\right) = 29\%$$

$$\Phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{4.86-1}{4.86}\right) = 20\%$$

$$\Phi_{C1} = 100\% - \Phi_E = 100\% - 29\% = 71\% \text{ of head}$$

$$\Phi_{D1} = 100\% - \Phi_D = 100\% - 20\% = 80\% \text{ of head}$$

Correction of Φ_{C1} for floor thickness $= +\frac{t}{d}(\Phi_{D1} - \Phi_{C1}) = \frac{0.6}{1.5}(80 - 71) = +3.6\%$

Correction of Φ_{C1} due to interference of D/S cutoff

$$C = \pm 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = +19\sqrt{\frac{1.3}{12}}\left(\frac{0.9+1.3}{13}\right) = +1.0\%$$

$\therefore \Phi_{C1}$ corrected = (71+3.6+1.0) % = 75.6% of head

b. Downstream Cutoff

d=1.7 m, b=13 m

Assume D/S floor thickness near cut off = 0.6 m (Q > 1.5 m^3/sec)

$$\alpha = \frac{b}{d} = \frac{13}{1.7} = 7.65$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+7.65^2}}{2} = 4.35$$

$$\Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{4.35-2}{4.35}\right) = 32\%$$

$$\Phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{4.35-1}{4.35}\right) = 22\%$$

$$\Phi_{C1} = 0\%$$

Correction of Φ_E for floor thickness $= -\frac{t}{d}(\Phi_E - \Phi_D) = -\frac{0.6}{1.7}(32 - 22) = -3.5\%$

Correction of Φ_E due to interference of U/S cutoff

$$C = \pm 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = -19\sqrt{\frac{0.70}{12}}\left(\frac{1.1+0.70}{13}\right) = -0.6\%$$

$\therefore \Phi_E$ corrected = (32 - 3.5 - 0.6) % = 27.9% of head

6. Floor thickness

a. D/S Floor

1. At 2 m From D/S End

$$\%\text{pressure} = 27.9 + \left(\frac{75.6-27.9}{12}\right) \times 1.5 = 33.86\% \text{ of head}$$

Minimum concrete floor thickness $t_{min} = \frac{0.3386\times 2.2}{2.4-1} = 0.53\ m < 0.6$

Provide 0.6 m thick concrete floor for 1.5 m length.

2. At 4 m From D/S End

$\%\text{pressure} = 41.8\%$ of head

Provide 0.7 m thick concrete floor for 2.0 m long.

3. At the toe of glacis (beginning of the hydraulic jump)

$$\%\text{pressure} = 27.9 + \left(\frac{75.6 - 27.9}{12}\right) \times 5.5 = 49.8\% \text{ of head}$$

Unbalanced head = 0.498 * 2.2 = 1.095m

Unbalanced head due to dynamic condition = 50% ($D_2 - D_1$) + $\Phi_{at\ toe}$ * HL

= 0.5× (1.32 -0.32) +0.498×0.8 = 0.898 m < 1.095

$$\therefore t_{min} = \frac{1.095}{2.4-1} = 1.39 \text{ m}$$

Provide floor thickness 1.4 m.

b. U/S Floor Thickness

Same as provided in U/S floor for the cross regulator (minimum thickness of 0.6 m in the U/S which should be thickened under the crest).

Chapter Five

Design of Pipes and Box Culvert

5.1. Introduction

A transverse structure is built under the road to transport the water from side to another (Figure 5.1).

Figure (5.1): Box Culvert

5.2. Design Criteria

The operating head, i.e. the difference of the upstream and downstream levels (say $\mathbf{H_L}$) will give us the maximum head causing flow. The area of the opening should then be decided, so that it is sufficient to pass the design discharge (Q).

If **V** is the velocity through the culvert opening, **which runs full in such a case**, then the head loss ($\mathbf{H_L}$) will be equal to:

1. The entrance loss ($\mathbf{h_e}$) $= K_e \frac{V^2}{2g}$ 2. Friction loss in the barrel ($\mathbf{h_f}$) 3. The outlet loss ($\mathbf{h_o}$) $= K_o \frac{V^2}{2g}$

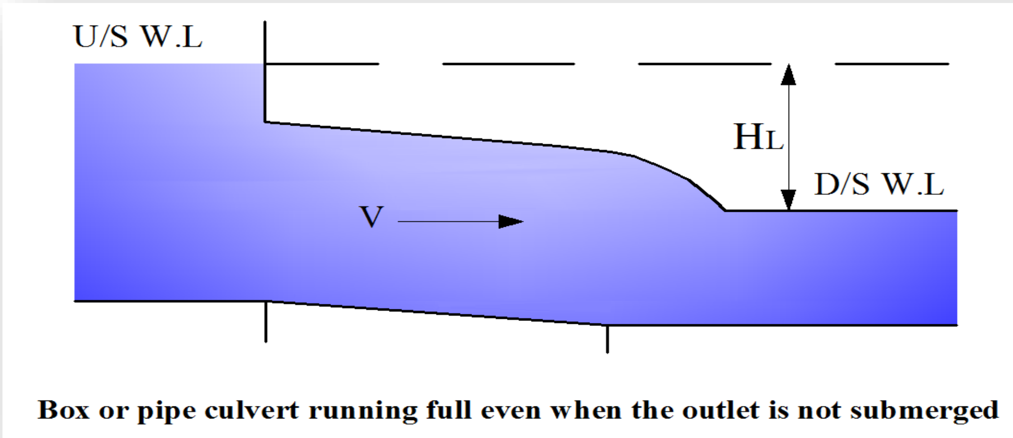


Figure (5.2): Box or Pipe Culvert running Full even when the Outlet is not Submerged

Note: The entrance & outlet losses depends on the type of the shape provided for entrance & outlet, and it may be:

1. Square edged entrance ($K_e = 0.505$)
2. Well rounded entrance ($K_e = 0.05$)
3. Curvature entrance ; if $\frac{r}{D} = 0 \quad \Rightarrow K_e = 0.1$

   $\frac{r}{D} = 0.05 \Rightarrow K_e = 0.25$

4. For grooved or socket end pipe, the value of $K_o = 1$
5. Losses in bent $h_b = K_b * \frac{V^2}{2g}$

K_b can be calculated as shown in the table (5.1).

Table (5.1): Values of Minor Losses Coefficient for Bents

R/D	90^o	45^o	22.5^o
≥ 5	0.6	0.45	0.3
4.3	0.65	0.5	0.35
2	0.75	0.55	0.4
1	1	0.75	0.5
Elbow	0.7	---	----
T	1.8	---	----

The friction loss $\mathbf{h_f}$ can be calculated by Manning's formula as:

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

$$h_f = S \times L$$

L: length of culvert

If the flow in the pipe is full flow then

R = D/4; where **D**: diameter of the pipe

$$\therefore h_f = \frac{n^2 V^2 L}{\left(\frac{D}{4}\right)^{4/3}}$$

∴ $\mathbf{H_L}$ = Entrance loss + Friction loss + Outlet loss

$$H_L = \left[K_e \frac{V_1^2}{2g} + \frac{n^2 V^2 L}{\left(\frac{D}{4}\right)^{4/3}} + K_o \frac{V^2}{2g}\right] = [K_e + K_f + K_o]\frac{V^2}{2g}$$

$$K_f = \frac{2g\, n^2 L}{\left(\frac{D}{4}\right)^{4/3}}$$

Hence, knowing the area, the discharging capacity of the culvert, or by knowing the discharge, the required area can be easily computed. The required sized pipe (or a rectangular barrel) may then be constructed through the road embankment.

Example 5.1: Design a box culvert for the given data:

$Q = 43.8\ m^3/sec$; length of culvert (L) = 50 m; $K_e = 0.3$; $K_o = 0.6$; n = 0.015; operating head (H_L) = 0.4 m. If the inlet height of the culvert does not exceed 3 m.

Solution:

$$H_L = [K_e + K_f + K_o]\frac{V^2}{2g}$$

$$H_L = \left[K_e + \frac{2g\,n^2\,L}{\left(\frac{D}{4}\right)^{4/3}} + K_o\right]\frac{Q^2}{A^2\,2g} \quad ; \quad Q = AV \quad \text{and} \quad A = D*D$$

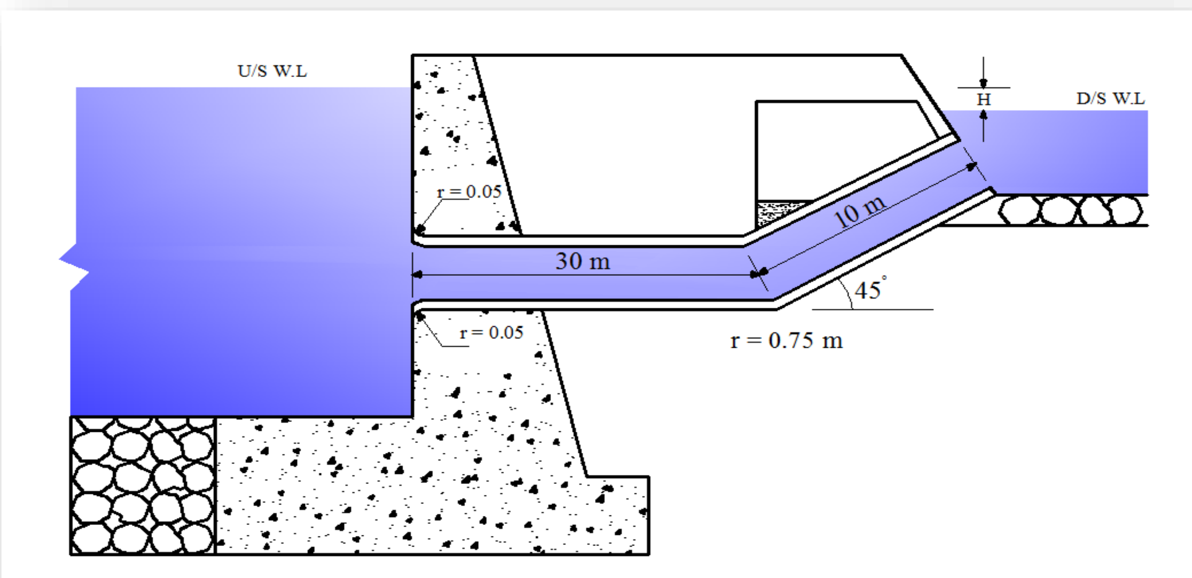
$$0.4 = \left[0.3 + \frac{2\times9.81\times0.015^2\times50}{\left(\frac{D}{4}\right)^{4/3}} + 0.6\right]\frac{43.8^2}{D^4\,2\times9.81} \quad \Rightarrow D = 4\text{ m (By trial \& error)}$$

But the max. height of culvert equal to 3.0 m,

∴ use a rectangular box

$$A = D^2 = b\times h = 16 \qquad \Rightarrow \qquad b = \frac{A}{h} = \frac{16}{3} = 5.33\text{m}$$

Example 5.2: A precast concrete pipe culvert is to be constructed under road as sketched below. Find the head drop (H) that takes place with a discharge of 0.5 cumec and a pipe diameter of 0.75 m.



Solution:

For inlet k_e 0.25 (approx.) because $r/D = \frac{0.05}{0.75} = 0.066$

For outlet $k_o = 1$

For bend $R/D = \frac{0.75}{0.75} = 1.0 \quad \theta = 45 \quad \Rightarrow$ from table $K_b = 0.75$

For friction $k_f = \frac{124.58\, n^2 L}{D^{4/3}} = \frac{124.58 \times 0.015^2 * 40}{(0.75)^{4/3}} = 1.65$

$$\sum K = 0.25 + 1.0 + 1.65 + 0.75 = 3.65$$

$$H = \sum K \frac{V^2}{2g} \quad , \quad H = 3.65 * \frac{(0.5)^2}{2*9.81} * \frac{1}{\pi^2 (\frac{0.75^2}{4})^2} = 0.24m$$

H.W.1: A concrete pipe culvert is to be constructed under road to carry a maximum discharge 9 m^3/sec. If the length of pipe culvert 10 m on slope 1/100. Find the diameter of the pipe and draw the culvert between the change of discharge and the diameter.

Chapter Six

Design of Inverted Siphon

6.1. Introduction

Inverted siphons are used to convey canal water by gravity under roads, rail roads, other structures, various types of drainage channels and depressions. A siphon is a closed conduit designed to run full and under pressure. Inverted siphons (sometimes called sag culverts or sag lines). The structure should operate without excess head when flowing at design capacity (Figure 6.1).

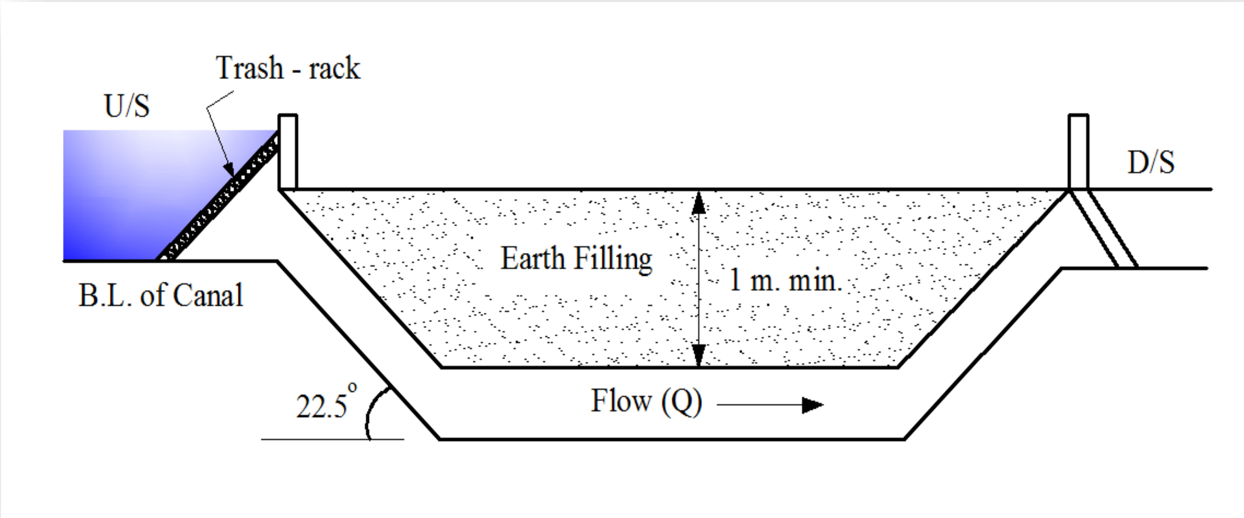


Figure (6.1): Inverted Siphon Details

6.2. Application

Economics and other considerations determine the feasibility of using an inverted siphon or another type of structure. The use of an elevated flume (aqueduct) would be an alternative to an inverted siphon crossing such features as a deep roadway cut or another channel. The use of a raised grade line and culvert may be a more economical alternative to employing a siphon under a road.

6.3. Hydraulic Design Consideration

The size of siphon is a function of head, velocity and economy for the pipe. The following steps may take into consideration during the process of design of siphon:

1. Assume internal dimension for the siphon. Note that when Q < 2.5 m^3/sec, use round section and when Q > 2.5 m^3/sec, use square section.
2. Compute the total head losses which consist of:
   a. Entrance and outlet.
   b. Head loss in the two screens (trash rack).
   c. Head loss due to friction in the barrel.
   d. Head loss in the two elbows.
3. Energy grade line elevation is the differences between U/S and D/S = the sum of all computed losses.

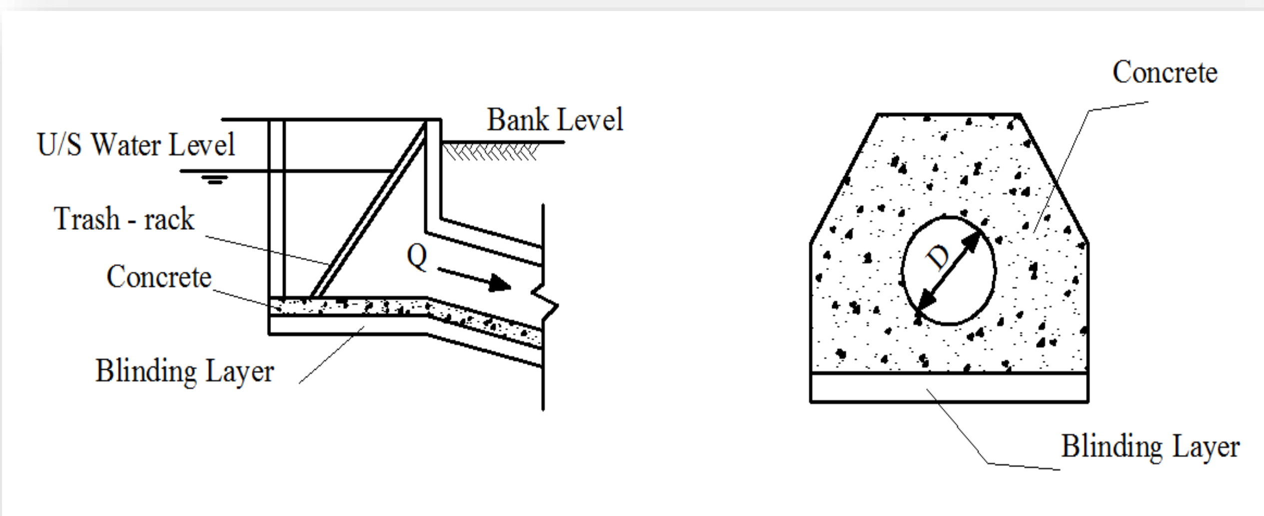
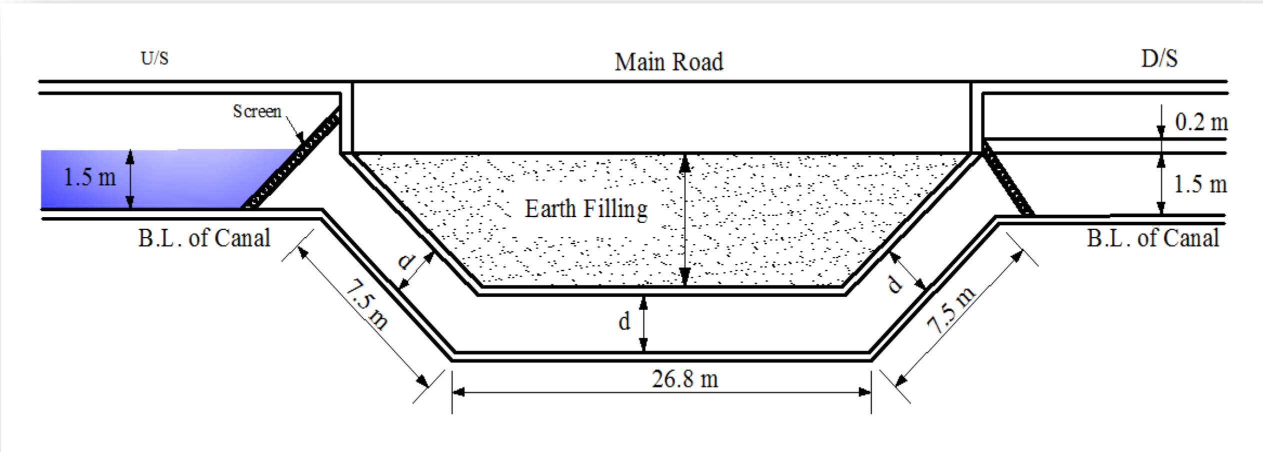


Figure (6.2): Side and Front View for the Entrance

Example 6.1: **Design an inverted siphon required to pass a canal discharge of 4.0 m^3/sec under main road with 0.2m head loss. The velocity in the canal is 0.82 m/sec and the depth of water in the canal 1.5m. Safety screen are provided of entry and exit. The inverted siphon has 22.5° elbows of each end. The site dimension are shown as below. (Take n= 0.013, K_e= 0.2, K_o= 0.3, $K_{screens}$= 0.2, K_{elbows}= 0.05)**



Longitudinal Section of Siphon (L - Section)

Solution:

Total head loss = 0.2m

$\Delta H = 0.2m = U/S\,(W.L) - D/S\ (W.L) = \text{sum of the losses}$

For discharging more than 2.5 m^3/sec , use square section.

$Q = 4\ m^3/sec$, hence use square section.

$A = d^2 \qquad P = 4d \qquad R = \frac{A}{P} = \frac{d}{4}$

Velocity in the canal = 0.8 m/sec

Velocity head $= \frac{V^2}{2g} = \frac{0.82^2}{2*9.81} = 0.034m$

Discharge in the barrel $Q = 4\ m^3/sec$

Assume velocity in the barrel = V_1

$V_1 = \frac{Q}{A} = \frac{4}{d^2}\,m/sec$

Velocity head$=\frac{V_1^2}{2g}=\frac{16}{2gd^4}$

Length of the barrel = $7.5+26.8+7=41.3$ m

1. Friction losses in the barrel

$$h_f=\frac{2gn^2L}{R^{4/3}}*\frac{V_1^2}{2g}==\frac{n^2*L}{(d/4)^{4/3}}*V_1^2=\frac{(0.013)^2*41.3*(4)^{4/3}}{(d)^{4/3}}*\frac{16}{d^4}=\frac{0.709}{d^{5.33}} \qquad \ldots(1)$$

2. Entrance & outlet losses:

$$=K_e\left(\frac{V_1^2}{2g}-\frac{V^2}{2g}\right)+K_o\left(\frac{V_1^2}{2g}-\frac{V^2}{2g}\right)=0.2\left(\frac{V_1^2}{2g}-\frac{V^2}{2g}\right)+0.3\left(\frac{V_1^2}{2g}-\frac{V^2}{2g}\right)$$

$$=\left(\frac{16}{2gd^4}-0.034\right)=\left(\frac{0.4077}{d^4}-0.017\right) \qquad \ldots(2)$$

3. Head loss in two elbows:

$$=2*0.05*\frac{V_1^2}{2g}=2*0.05*\frac{16}{2*9.81*d^4}=\frac{0.08155}{d^4} \qquad \ldots(3)$$

4. Head loss in two screens

$$=K_{screen}\frac{V^2}{2g}=(0.2+0.2)\frac{V^2}{2g}=0.4*\frac{0.82^2}{2*9.81}=0.0137 \qquad \ldots(4)$$

$\therefore$ Total loss = sum of Eq. $(1+2+3+4)=0.2$m

$$0.2=\frac{0.709}{d^{5.33}}+\left(\frac{0.4077}{d^4}-0.017\right)+\frac{0.08155}{d^4}+0.0137$$

$$0.2=\frac{0.709}{d^{5.33}}+\frac{0.4893}{d^4}+0.0307$$

By Trail & Error $\Rightarrow$ $d=1.5$ m

$\therefore$ Use dimension of Box $=1.5*1.5$ m

Chapter Seven

Weirs

7.1. Definition

An overflow structure designed to measure the discharge of water in a river or open channel. It is placed perpendicular to the flow direction of water. It is also used to prevent flooding or to make a river more navigable.

7.2. Practical Purposes of Weirs

Weirs are used for the following purposes:

1. To maintain high water level in order to divert water into a diversion channel for irrigation or Power purpose (Figure 7.1).

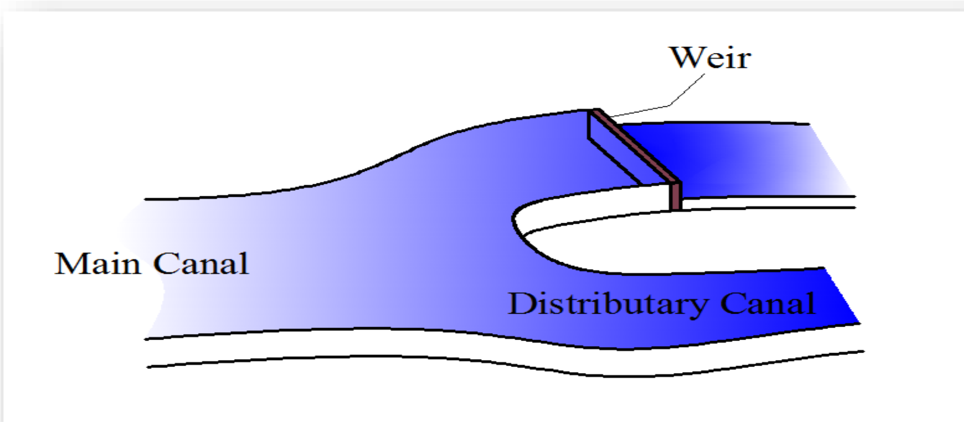


Figure (7.1): Raising Water Level in the Main Canal

2. To gauge the discharge of branch channel at their intakes, the discharge of drains at their escape & the discharge of canals funding power houses.

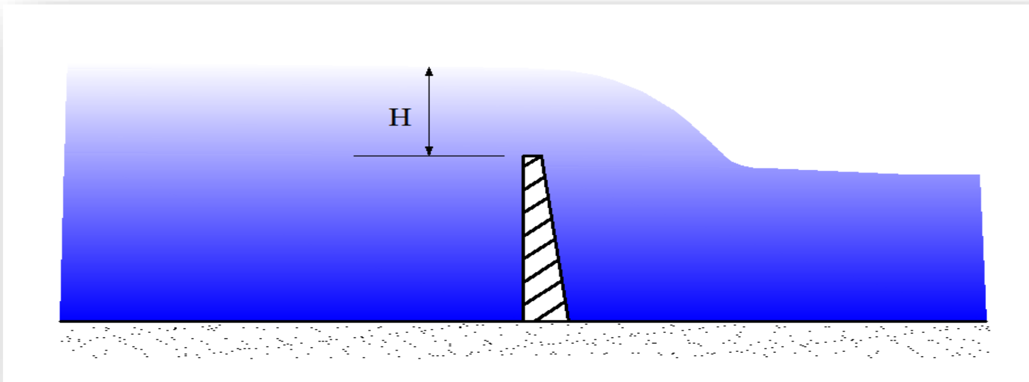


Figure (7.2): Head of Water above the Crest of Weir

3. Water can be stored for a short period

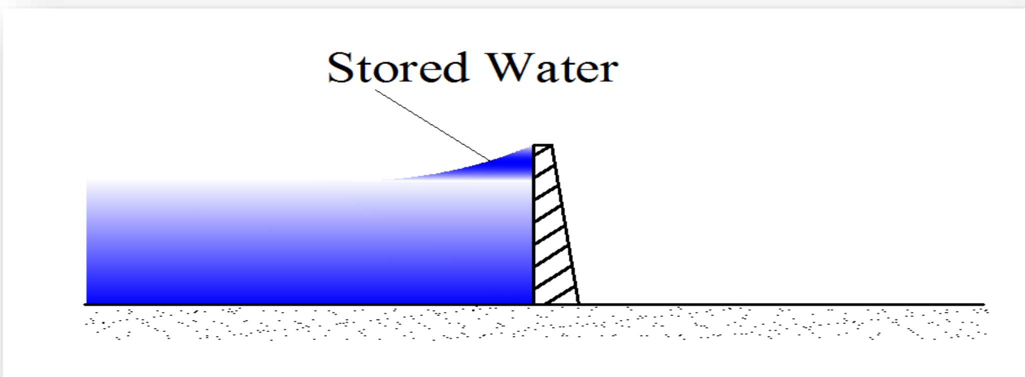


Figure (7.3): Stored Water Location

4. To reduce the head acting on a barrage.

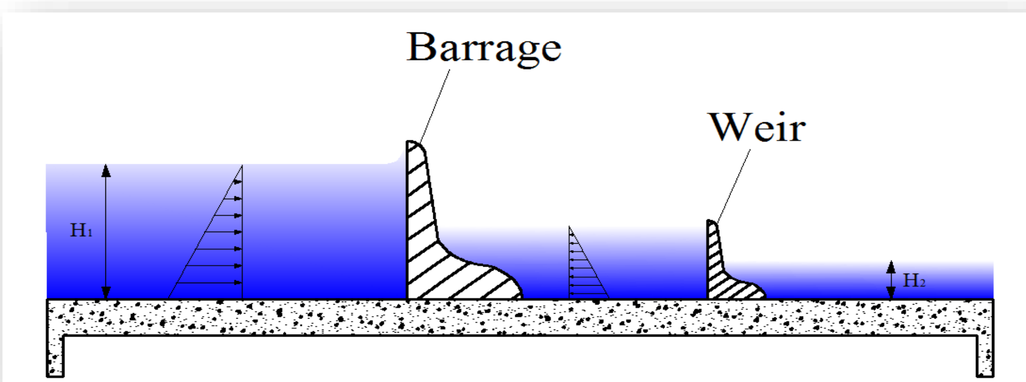


Figure (7.4): Reducing Head after Barrages

5. To reduce the water slope in case of a very steep land.

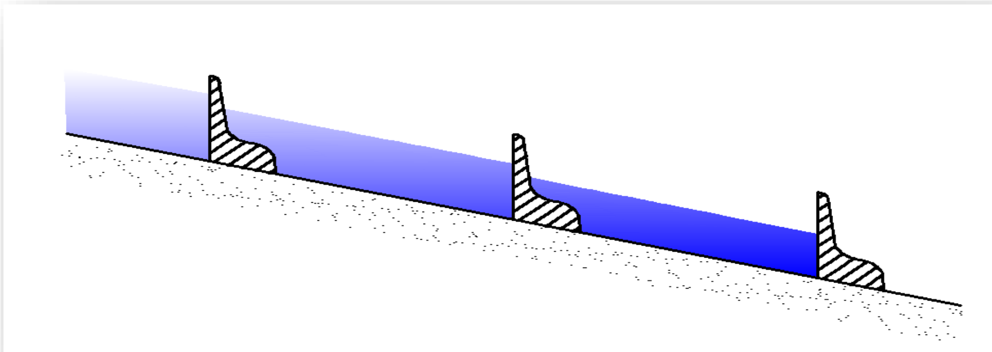


Figure (7.5): Reducing Head by the Effect of Successive Weirs in Steep Slopes

6. To escape the water in canal automatically.

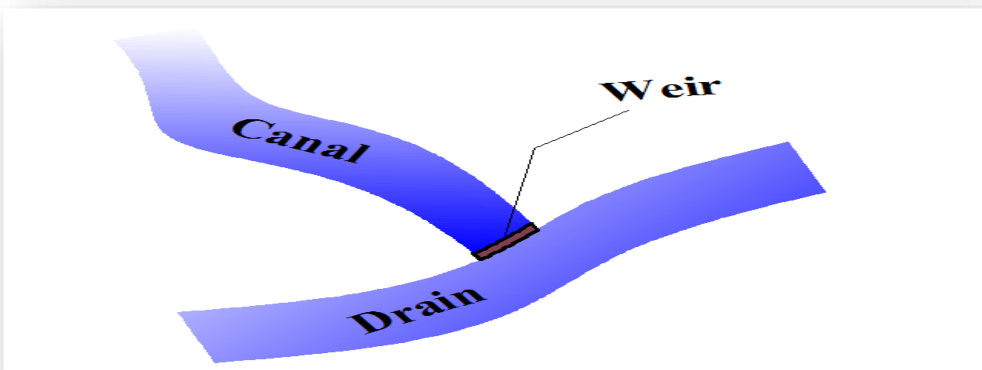


Figure (7.6): Regulating the Escape of Water from Canals

7. To control silt movement into the canal system, we can use the weir to many purpose at the same time.

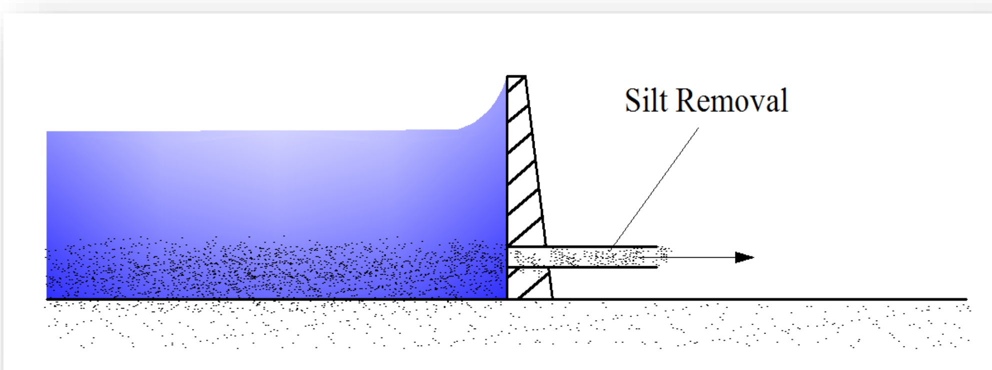


Figure (7.7): Silt Removal

Weirs are good flow measuring devices. They are commonly constructed as:

1. Sharp – Crested Weir: with opening of the shapes :

a. Rectangular b. Triangular c. Trapezoidal d. circular

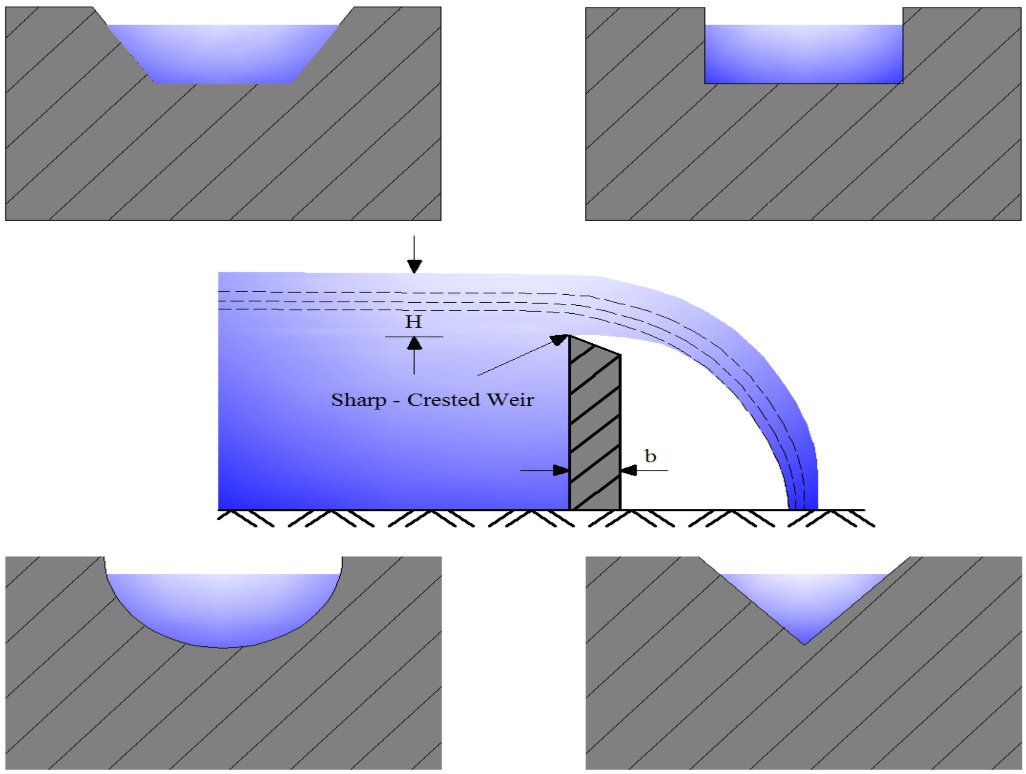


Figure (7.8): Shapes of Sharp Crested Weirs

Sharp – Crested weirs also classified as:

a. Contracted weirs: where the width of the channel is greater than the width of the weir opening.

b. Suppressed weirs: where the width of the channel and the width of the weir opening are equal.

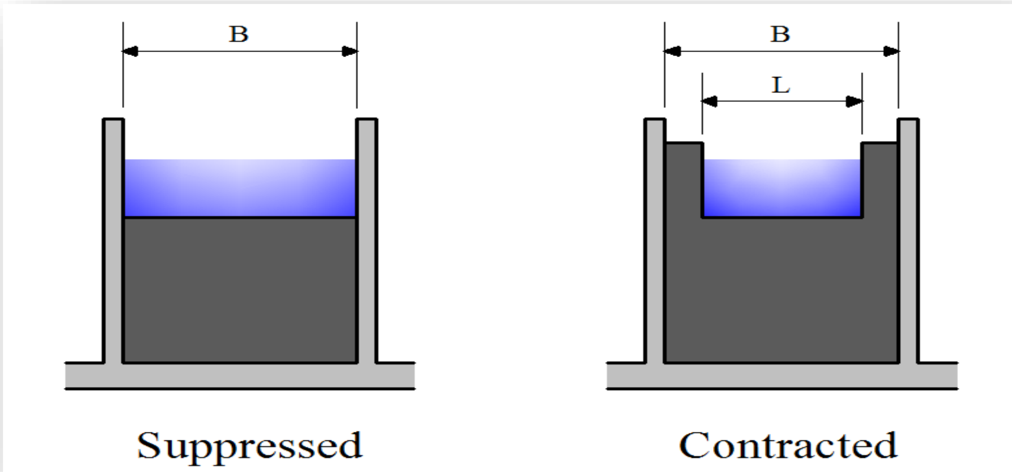


Figure (7.9): Suppressed and Contracted Sharp Crested Weirs

2. Broad – Crested Weir :

Where the flow is significantly influenced by viscous drag which is enumerated in the form of a discharge – coefficient.

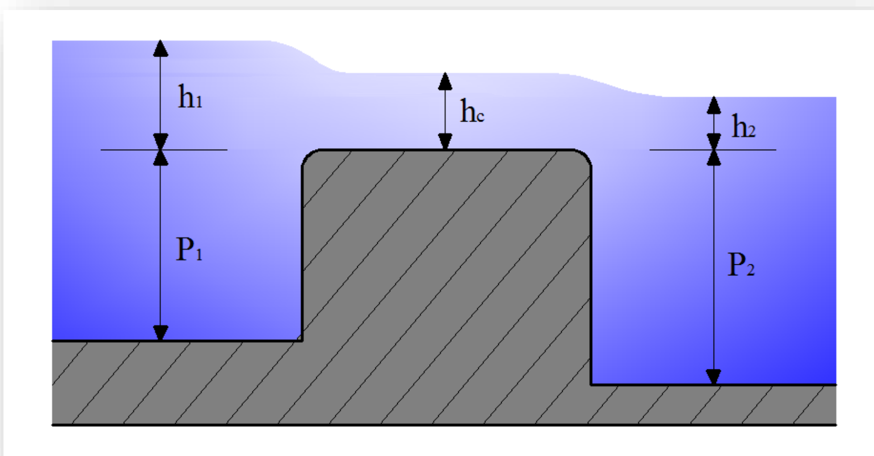


Figure (7.10): Broad Crested Weirs

7.3. Hydraulics and Theories of Weirs

1. Rectangular Sharp – Crested Weirs

Hydraulic equation of this type can be simplified to the theoretical form as:

$$q = \frac{2}{3}\sqrt{2g}\,H^{3/2} \qquad \dots (1)$$

For real weir flow

$$q = C_d \cdot \frac{2}{3}\sqrt{2g}\,H^{3/2} \qquad \dots (2)$$

Where **C_d** is the coefficient of discharge.

The experimental work of Rehbock led to an empirical formula for **C_d** of well-ventilated sharp- crested rectangular weir:

$$C_d = 0.611 + 0.08\frac{H}{P} \qquad \text{if } \left(\frac{H}{P} \leq 5\right)$$

The effective width (b) is considered as:

1. b = B (for suppressed rectangular weir)
2. b = (B – n× 0.1 H) (for contracted rectangular weir)

n: number of contractions (usually one to each side)

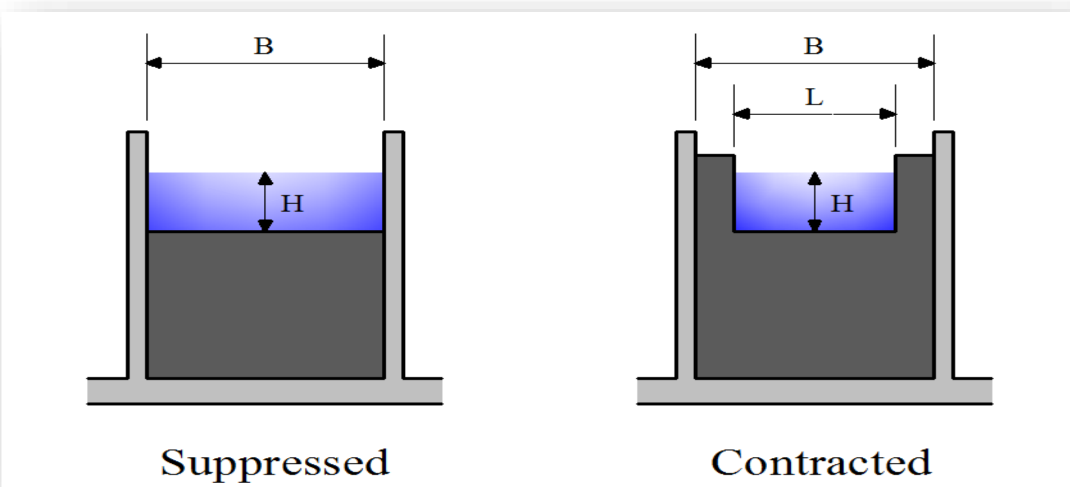


Figure (7.11): Rectangular Sharp Crested Weirs

2. Triangular Weir (V- notch)

This type is widely used as measuring device for small flow rates.

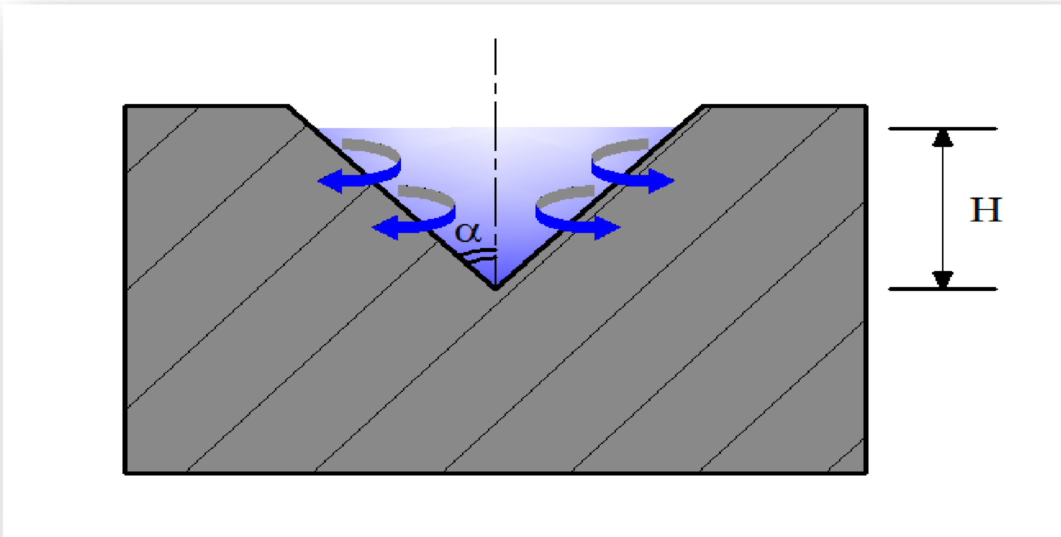


Figure (7.12): Triangular Sharp Crested Weir

A simplified analysis yield the fundamental formula as:

$$Q = C_d * \frac{8}{15} \tan\alpha \sqrt{2g} \; H^{5/2} \quad \ldots (3)$$

Coefficient (C_d) for Lenz is:

$$C_d = 0.56 + \frac{0.7}{R_e^{0.165} \, W^{0.17}}$$

R_e: Reynolds No.

W: Surface tension

The conditions of (C_d) for Lenz:

1. $H > 0.06$ m 2. $R_e > 300$ 3. $W > 300$

<u>**Note:**</u> $C_d \approx 0.59$ for weir of ($2\alpha = 90°$)

3) Trapezoidal Weir (Cipolletti Weir) with (S.S 1:4)

This can be considered as one rectangular notch of width (b) and two half V-notch (angle = α =14). The discharge equation may be written as:

$$Q = C_{d1}\,\frac{2}{3}\,\sqrt{2g}\,(b - 0.2H)\,H^{3/2} + \frac{8}{15}\,C_{d2}\,\sqrt{2g}\,\tan\propto\,H^{5/2} \quad \text{... (4)}$$

due to presence of 2 ends contractions.

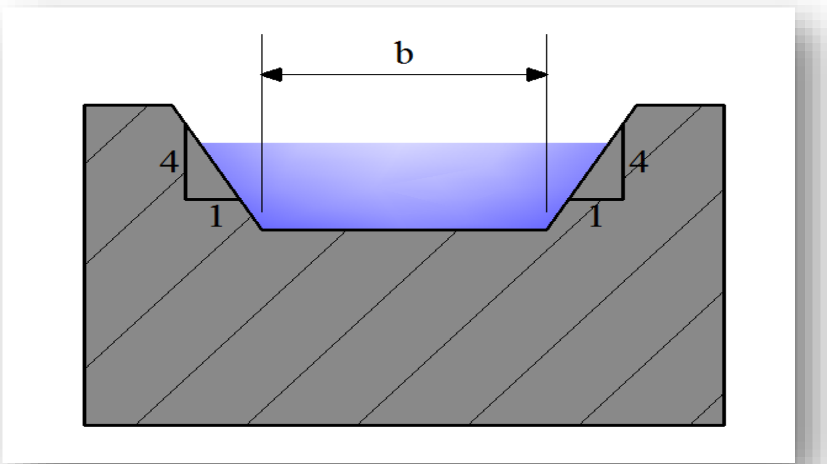


Figure (7.13): Trapezoidal Sharp Crested Weir

When we use this type of weir , we can obtained W.L more stability than The type of rectangular Weir because that (b) increase with increase of the depth & give a greater discharge & keep the W.L At stable , therefore ; its use in the escape weir.

Example 7.1: Determine the discharge over a sharp crested weir 4.5m long with no lateral constrictions (suppressed). The measured head over the crest being 0.45m and the sill height of the weir is 1m.

Solution:

$$Q = C\frac{2}{3}\,b\,\sqrt{2g}\,H^{3/2}$$

$\frac{H}{P} = \frac{0.45}{1.0} = 0.45 < 5$

Cd =0.611+ 0.08 $\frac{H}{P}$ = 0.611 + 0.08 $\frac{0.45}{1}$ = 0.647

$Q = 0.647 * \frac{2}{3} * 4.5\sqrt{2 * 9.81}(0.45)^{3/2} = 2.61\ m^3\ /sec$

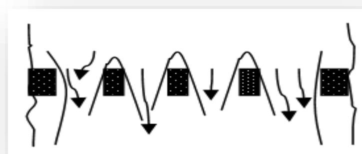
Example 7.2: A 6m long weir was measured to carry a 1.4 m^3/ sec discharge when the crest is over topped by 0.2m of water. Determine the discharge coefficient of the weir?

Solution:

$C_d = \frac{Q}{\frac{2}{3} b\sqrt{2g}\, H^{3/2}} = \frac{1.4}{\frac{2}{3}*6*\sqrt{2*9.81}*(0.2)^{3/2}} = 0.883$

Example 7.3: A 30 m long weir is divided into 10 equal bays by vertical posts each 0.6 m wide. Calculate the discharge over the weir under an effective head of 1m? C_d = 0.623

Solution: Sometimes the total length of a weir is divided into a number of bars or span by vertical posts in such case, the number of bays or spans, into which the weir is divided.



No. of bays = 10 (30 m length of weir)

Width of each post = 0.6 m

Effective length L = (30 - 9* 0.6) = 24.6m

No. of end contractions, n = 2*10 = (one bay has two end contraction)

$Q = \frac{2}{3}\ Cd\ (L - 0.1nH)\sqrt{2g}\ H^{3/2}$

$= 2/3 * 0.623 * \sqrt{2g}\ (24.6 - 0.1*20*1) * 1^{3/2} = 41.6\ m^3/sec$

7.4. Design of Escape Weir

1. Rectangular Escape Weir

Let **AB** be the normal water line in the canal which should be ended at a natural drain of the point **B**.

To avoid a sudden drop in water a weir is constructed (**Escape Weir**). For the determination of the dimensions of this weir, knowing (alternatively) either **b** or **p** the remaining dimension (p or b) can be directly obtained from the weir formula:

$$\mathbf{Q = Cd * \frac{2}{3}\sqrt{2g}\, H^{3/2}}$$

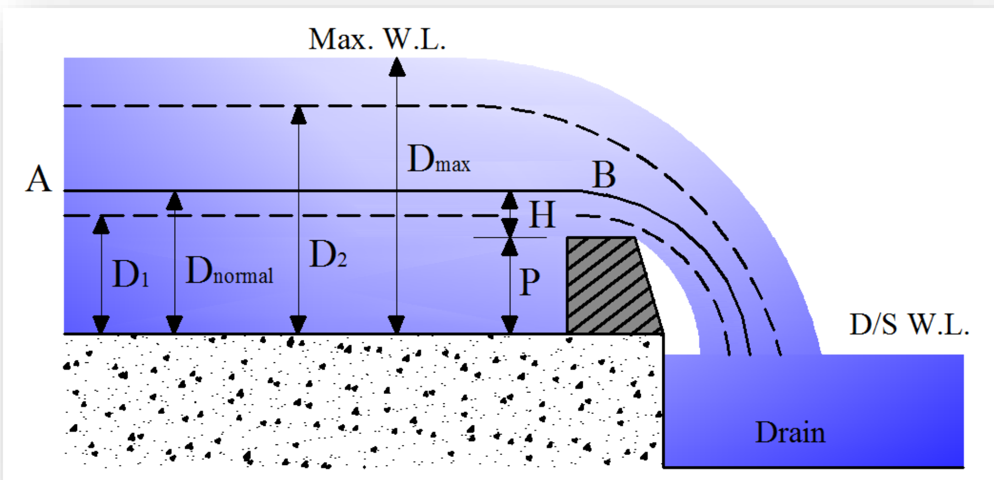


Figure (7.14): Escape Weir

Regarding that

P = D – H or H = D – P

As Q & H are given

On the other hand, knowing both the canal properties & the range of max. & min. water depth (D_{max}, D_{min}). Then two cases at four D ranges can be chosen:

$D_1 = D_{min} + \frac{1}{4}\,(D_{max} - D_{min})$

$D_2 = D_{min} + \frac{3}{4}\,(D_{max} - D_{min}) \implies$ or $D2 = Dmax - \frac{1}{4}\,(D_{max} - D_{min})$

Now, if the corresponding discharges for D_1 & D_2 are Q_1 & Q_2 and corresponding depth over sill are:

H_1 & H_2

$$Q_1 = Cb\, H_1^{3/2} = Cb\,(D_1 - P)^{3/2} \qquad \ldots (1)$$

$$Q_2 = Cb\, H_2^{3/2} = Cb\,(D_2 - P)^{3/2} \qquad \ldots (2)$$

Where $C = Cd\,.\frac{2}{3}\sqrt{2g}$

From which **b** & **P** are found.

Example 7.4: Given an open channel of the following properties:

Min water depth = 1m (0.7 Q)

Max water depth = 2.5m (1.2Q)

Bed width = 2m

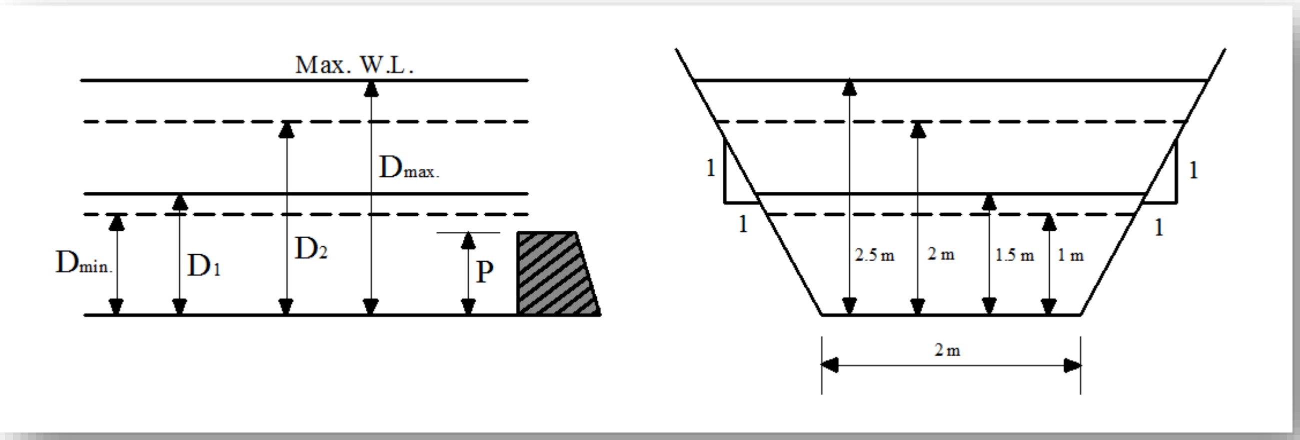
Bed slope = 15 cm/km

Side slope = 1:1

Manning's roughness = n = 0.03

It is required to design an escape weir at the end of this channel assuming that: $C = \frac{2}{3}\, Cd\sqrt{2g} = 1.6$

Solution:



$D_1 = D_{min} + \frac{1}{4}\ (D_{max} - D_{min}) = 1 + \frac{1}{4}\ (2.5 - 1) = 1.375$

$D_2 = D_{min} + \frac{3}{4}\ (D_{max} - D_{min}) = 1 + \frac{3}{4}\ (2.5 - 1) = 2.125$

Using the Manning's eq. With the canal properties:

$Q = \frac{1}{n}\ AR^{2/3}\ S^{1/2}$; $A = B*d + z*d^2$ or $(B + z*d)\ d$; $P = B + 2d(1+z^2)^{0.5}$

$$Q_1 = \frac{1}{0.03}(2 + 1 * 1.375\ ) * 1.375 * \left[\frac{(2+1.375*1)*1.375}{2+2*1.375\sqrt{2}}\right]^{2/3} * (0.00015)^{1/2}$$

$Q_1 = 1.62\ m^3/sec$

$$Q_2 = \frac{1}{0.03}(2 + 1 * 2.125) * 2.125 * \left[\frac{(2+1*2.125)*2.125}{2+2*2.125\sqrt{2}}\right]^{2/3} * (0.00015)^{1/2}$$

$Q_2 = 3.8\ m^3/sec$

Using the weir formula:

$Q_1 = Cb\ {H_1}^{3/2} = Cb\ (D_1 - P)^{3/2}$

$Q_2 = Cb\ {H_2}^{3/2} = Cb\ (D_2 - P)^{3/2}$

$1.62 = 1.6 * b * (1.375 - P)^{1.5}$ ……………(1)

$3.8 = 1.6 * b * (2.125 - P)^{1.5}$ ……………(2)

$(2.346)^{2/3} = \frac{2.125-P}{1.375-P}$

From which $P = 0.4$ m & then $b = 1.05$ m

---

2. Trapezoidal Notch – Fall Escape

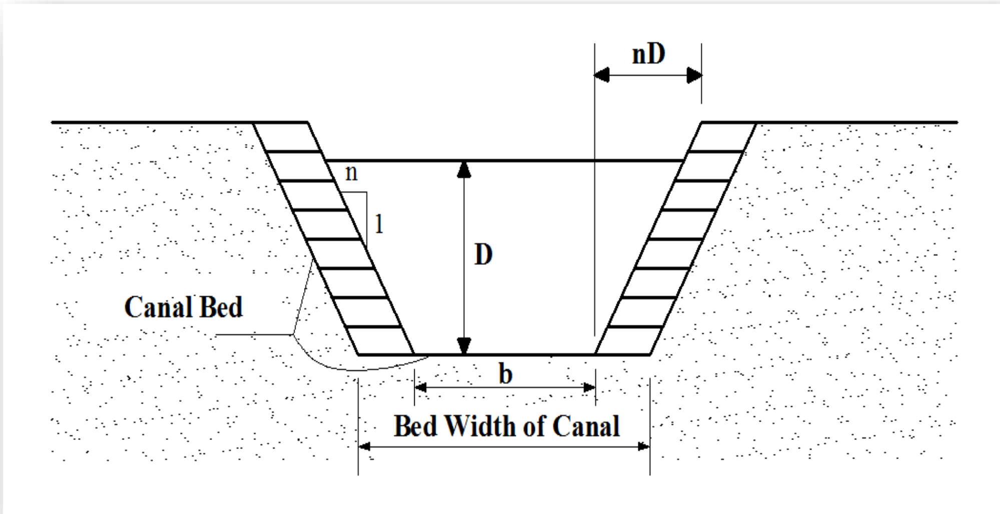
In the previous type of escape weir the canal cannot be drained totally unless a pipe is provided at bed level. This pipe adds a certain variable to the problem.

A solution which is simpler being to adopt a V – notch fall on which the inclination of the sides takes the place of the depth (p) as a variable:

$\tan\theta = \frac{nD}{D} = n$ or $n = \tan\theta$

Two values of Q & D must be known to determine the two unknowns (b & h) from an equation which may be derived as follows:

The equation of the trapezoidal weir was previously given as:



$$Q = C_1 \frac{2}{3}\sqrt{2g}(b - 0.2H)H^{3/2} + \frac{8}{15} C_2 \sqrt{2g}(\tan\theta)H^{5/2} \qquad \dots (1)$$

For very low rectangular weir:

$C1 = 1.06\ (1+P/H) \qquad P=0 \qquad \dots (2)$

$\quad = 1.06\ (1+0) = 1.06 \ \approx 1.0$

Assume $C_2 \approx 0.6$

Then the last equation may approximate in SI units as:

$$Q = 1 * \frac{2}{3}\sqrt{2 * 9.81} * b * H^{3/2} + \frac{8}{15} * 0.6 * \sqrt{2 * 9.81} * n * H^{5/2}$$

In which $n = \tan\theta$

Or : $Q = 2.95 * b * H^{3/2} + 1.417 * H^{5/2}$

Rearranging (out of brackets)

$Q = 2.95\ H^{3/2}\ (b + 0.48\ n\ H)$

But we have: P=0 no. sill

H = D

$Q = 2.95\ D^{3/2}\ (b + 0.48\ n\ D) \qquad \dots (3)$

Example 7.5: For the previous example it is required to design a trapezoidal notch escape?

Solution: From manning's equation it has been found that:

$D_1 = 1.5$ m ; $Q_1 = 1.91$ m3/sec

$D_2 = 2.0$ m ; $Q_2 = 3.364$ m3/sec

Using equation (3):

$Q = 2.95\, D^{3/2}\,(b + 0.48\, n\, D)$

$1.91 = 2.95 * 1.5^{3/2}\,(b + 0.48\, n * 1.5)$ ... (1)

$3.364 = 2.95 * 2^{3/2}\,(b + 0.48\, n * 2)$ ... (2)

From eq. (1) & (2)

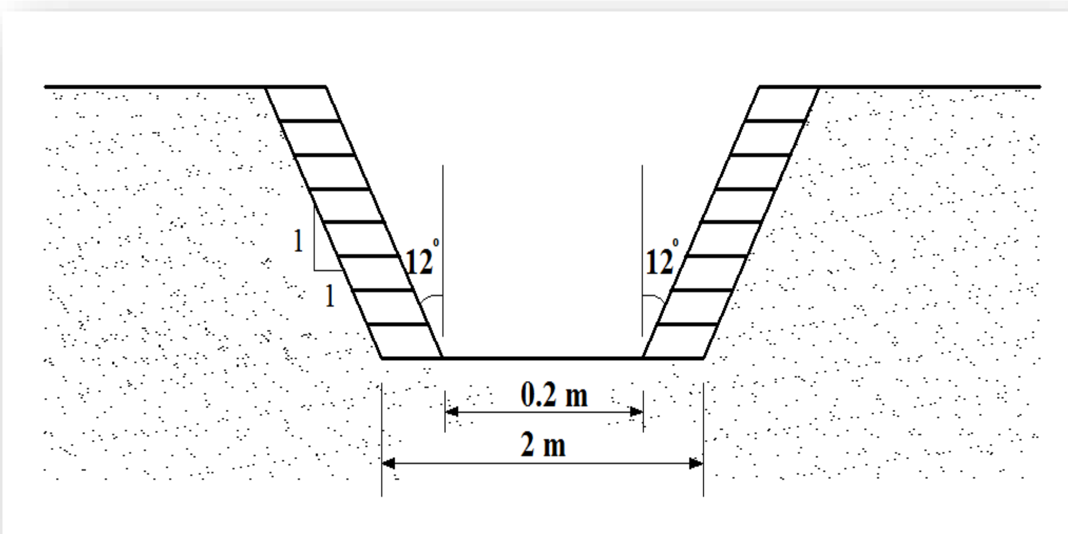
$b = 0.352 – 0.72\, n$ ... (1)

$b = 0.403 – 0.96\, n$ ... (2)

From which $n = 0.2125 = \tan\theta$

$\theta = 12^\circ$

& $b = 0.352 – 0.72 * 0.2125 = 0.2$ m



Chapter Eight

Design of Gates

8.1. Types of Gates

1. **Stop Logs:** are custom designed steel bulkheads that are used in channels or control structures to hold back the flow of water. They are individually lowered on top of one another via a vertical guide system to a pre-determined height, to suit the dimensions of the sluice that they are restricting the water flow through. The design of the Stop Logs is a single rectangular unit that can be used in single or modular format to hold back different volumes of water.

Figure (8.1): Stop Logs

2. **Vertical Gates:** It is used in small and large structures of width ranged between 1m to 8 m. There is a groove built in the both sides of channel to facilitate the movement of gate to the upward and downward directions.

Figure (8.2): Vertical Gates

The design of vertical gate would involve the design of the following:

1- Gate leaf
2- Groove embedment
3- heisting equipment

8.2. Principle for Design of Lifting Gate

1- The gate will act as slab support on two walls.
2- The total load was not uniform distributed but it various from top to bottom.
3- The total load transmitted to a stiffener member is equal to the area of loading may equal to the area of loading member.

The loading may be equally divided into **n** number (**n** part), each stiffener is located such that it carry a total load equal to the calculated

$$A = \frac{1}{2}\gamma_W h^2$$

$$a = \frac{1}{2n}y^2$$

8.3. Procedure for Design

1. Find the water depth.

- Front side water level (F.S.W.L)

- Bed level (B. L) under gate.

2. Height of gate (L) = water depth + 0.1 * water depth = 1.1 * d

3. Draw pressure diagram.

Pressure at any point= $\gamma_W *$ depth of water

4. Assume **n** division.

5. Find the location of stiffener from principle each.

6. Bending moment calculation.

Total load stiffener (beam) equal (W)

$W = \gamma_W * a * L$

- Stiffener is simply supported beam.
- The stiffener it can be angle, I beam or channel.

$$B.M = \frac{W.L}{8}$$

7. Find Z (The section modulus) $= \frac{M}{f_s}$

$$Z = \frac{I}{c} = \frac{bt^2}{6}$$

The plate as continuous slab$\left(\frac{W.L^2}{10}\right)$

8. Calculate the thickness of plate. Consider a unit of 1m width of slab spacing vertically & supported on the stiffener.

Find the max. Bending moment

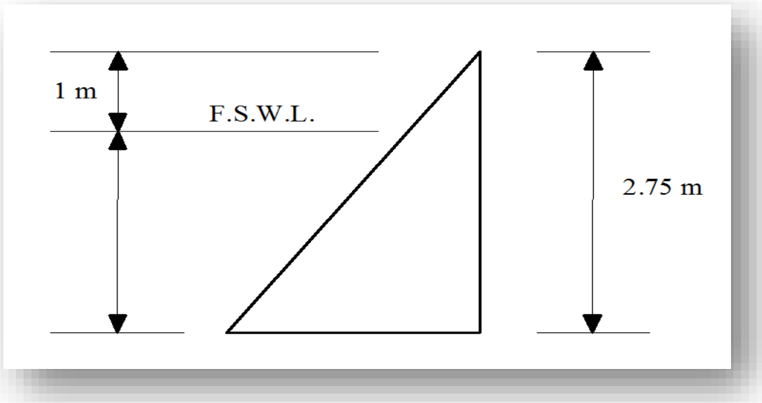
$$M = f_s * Z$$

$M_{max.} = f_s * \frac{bt^2}{6}$ Find t

Example 8.1: Design a vertical gate with the following information:

F.S.W.L = 35.25 m, Crest Level = 32.75 m, Width of Gate = 3 m, fs = 20000 psi

Solution:



Water depth = F.S.W.L – Crest level = 35.25 – 32.75 = 2.5 m

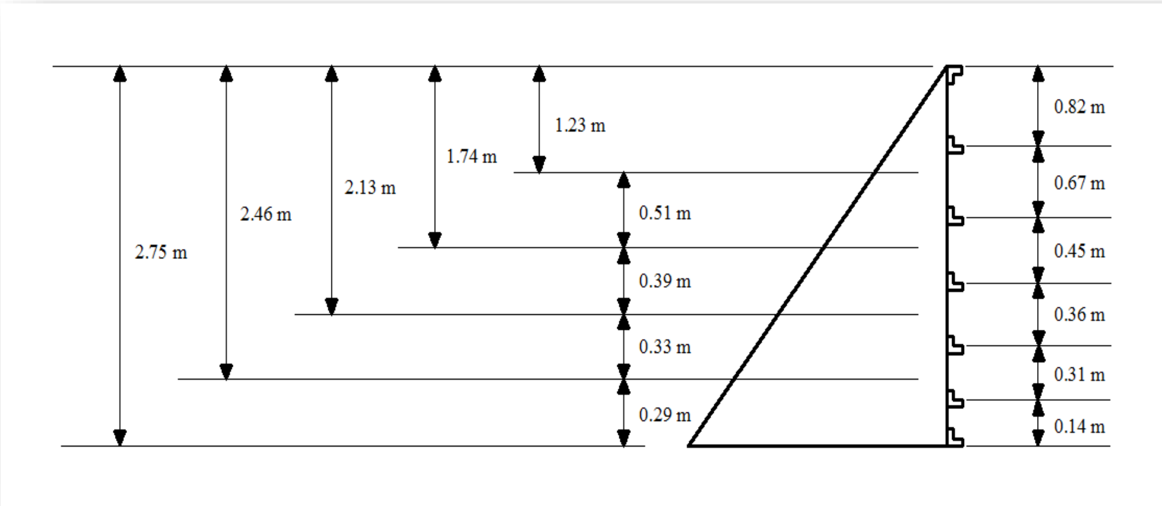
Height of gate = 1.1 * 2.5 = 2.75 m

Assume the number of division (n) = 5

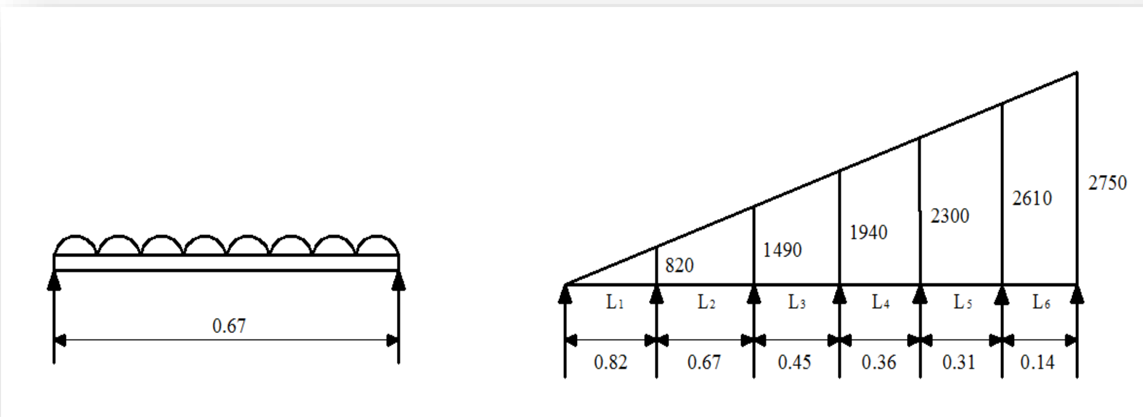
$a = \frac{1}{2n} y^2 = \frac{1}{2*5}(2.75)^2 = 0.756 \text{ m}^2$

$a_1 = \frac{y_1^2}{2} = 0.756 \rightarrow \quad y_1 = 1.23$ m

Area	Formula	Value of y	Division Height
0.756	$\frac{1}{2} y_1^2$	$y_1 = 1.23$	1.23
1.512	$\frac{1}{2} y_2^2$	$y_2 = 1.74$	0.51
2.268	$\frac{1}{2} y_3^2$	$y_3 = 2.13$	0.39
3.024	$\frac{1}{2} y_4^2$	$y_4 = 2.46$	0.33
3.78	$\frac{1}{2} y_5^2$	$y_5 = 2.75$	0.29
			$\sum = 2.75$



Calculation of Plate Thickness



Consider a unit of 1.0 m

Width of slab spacing vertically & supporting on the stiffeners.

Loads on span L_1, L_6 are about half strip

L_2, L_3 , L_4 , L_5 are equal & about (1/5) of the total area.

Assume an average uniform load on each strip, the total load (w) on each of them is the same.

The biggest moment will be the one with largest span.

Note: For accurate solution use moment distribution for determining (B.M.).

$W = 0.67 * \left(\frac{820+1490}{2}\right) = 775\ kg$

$M = \frac{W\,l}{10} = \frac{775*0.67}{10} = 52\ kg.m$ = 4580 Ib.in

B.M (Ib .in) = B.M (T.M) * 88000

Use fs = 20000 psi

Section modulus = $Z = \frac{M}{fs}$ … (1)

$Z = \frac{b\,t^2}{6} = \frac{40\,t^2}{6}$ … (2)

B = 1 m = 40 inch

$M = 4580 = \frac{20000*40\,t^2}{6}$

$t = 0.185 = \frac{3}{16}$ inch

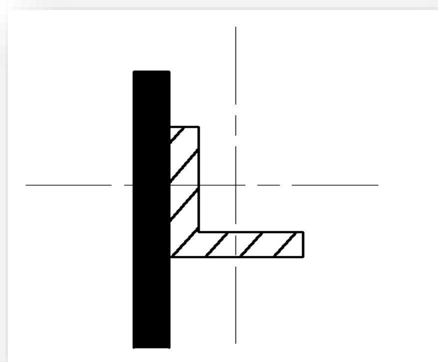
Design of Stiffeners (Beam Design)

Each stiffener is simply supported beam ($M = \frac{W\,L}{8}$)

Area of one strip = 756 kg/m

The stiffener can be angle, L beam

$756 * 3 = 2268\text{kg} = 5100\ \text{Ib}$



$$M = \frac{WL}{8} = \frac{5100*118}{8} = 75225$$

$$Z = \frac{M}{fs} = \frac{75225}{20000} = 3.76\ \ in^3$$

H.W: Design a sliding gate wing type of beams (w8*40) which have an elastic section modulus equal to 582 cm^3. If the height of gate 5m, span 3.5m and f_s =10000 T/m^2?

Chapter Nine

Dams

9.1. Introduction

The dam is a barrier constructed across the river to store water on its upstream side due to construction of a dam the water level of the river on its upstream is very much raised. Due to rise in water level large areas lying upstream of the dam get submerged. Dams are constructed to store the river water in form of on artificial lake or reservoir. The stored water can be utilized for generation of hydro-electric power, water supply, and irrigation or for any other purpose.

Figure (9.1): A Concrete Dam

9.2. Classification of Dams

Dams may be classified in several way as follows:

1. Classification Based on Materials of Construction

a. Earth fill dams

b. Rock fill dams

c. Concrete dams

d. Masonry dams

e. Steel dams

f. Timber Dams

2. Classification Based on Flow over its Top

a. Over flow dams

b. Non over flow dams

3. Classification Based on the Use of the Dam

a. Storage dams

b. Diversion dams

c. Detention dams

d. Multi-purposes dams

4. Classification Based on the Mode or Resistance Offered by the Dams against External Forces

a. Gravity dams

b. Buttress dams

c. Arch dams

5. Classification Based on Rigidity of the Dams

a. Rigid dams

b. Non- rigid dam

9.3. Advantages and Disadvantages of Gravity Dams

Advantages

1. Maintenance cost is negligible.
2. They are especially suitable for deep steep valley conditions where no other dam is possible.
3. If suitable foundation is available, such dams can be constructed for very large heights.

4. Because they can be constructed in very large heights, they can store more amount of water.
5. If suitable separate place is not available for installation of spillways, they can be installed in the dam section itself.
6. This dam gives prior indication of instability. If remedial measures are taken in time-unsafe dams may even be rendered safe. Even if they cannot be made safe they give sufficient time for the people to move out the area likely to be submerged due to failure of the dam.
7. Silting rate of the reservoir can be reduced considerably by installing under sluices in the dam near the bed of the reservoir. Sluices can be operated from time to time and silt may be scoured out of the reservoir.
8. They are not affected by very heavy rainfall. Earth dams cannot sustain very heavy rainfall because of heavy erosion.

Disadvantages

1. They are very costly in initial construction.
2. They take lot of time to construct.
3. They require skilled laborer for construction.
4. Such dams can be constructed only on good foundation.
5. If height of the dam is to be raised, it cannot be done unless provision for it had been made in the construction of the lower part of the dam.

9.4. Earth Fill and Rock Fill Dams

Advantage

1. They can be constructed on any type of foundation.
2. They can be constructed in comparatively less time.
3. They do not require skilled labor.
4. Initial cost of construction is low as locally available soils, and rock boulders are normally used.

5. Their height can be increased without any difficulty.
6. They are especially suitable for condition where slopes of river banks are very flat. Gravity dams under such conditions are not found suitable.

Disadvantage

1. The fail all of a sudden without giving any per-warning
2. Flood water affect the dam safety
3. Spillways have to be located independent of the dam
4. They cannot be constructed as over flow dams
5. They require continuous maintenance
6. They cannot be constructed in narrow steep valleys
7. They cannot with stand heavy rains unless properly protected
8. They cannot be constructed in large height. The usual height is 30 m for which most of the earthen dams

9.5. Factors Governing Selection Types of Dams

1. Topography

a. V-shape nor row valley select arch dam
b. Narrow U-shape valley indicates choice of over flow concrete dam
c. A low, rolling plan suggest earth dam

2. Geology & Foundation

a. Solid rock-foundation: select any type
b. Gravel &coarse sand foundation: select earth dam or Rock fill dam
c. Silt & fine sand foundation: select earth dam or low concrete dam up to 8m
d. Clay foundation: select earth dam with special treatment

3. Availability of Materials of Construction

a. If sand, gravel and stone is available , concrete gravity dam may be suitable
b. If coarse and fine grained soils are available, an earth dam may be suitable

4. Length and Height of Dam

If the length of the dam is very long and its height is low, an earth dam would be a better choice. If the length is small and height is more, gravity dam is preferred.

5. Spillways

a. Separate spillway lead to constructing earth dam

b. Large spillway with dam lead to concrete gravity dam and no separate

6. Road Way over the Dam

We can construct earth or gravity dam

7. Generation of Hydro-Electric Power

Concrete or masonry gravity dams because it can be constructed at height level and develop sufficient head for running the turbines.

9.6. Concrete Dams

Is a structure which is designed in such a way that its weight resist the force exerted up on it. It may constructed of concrete or masonry.

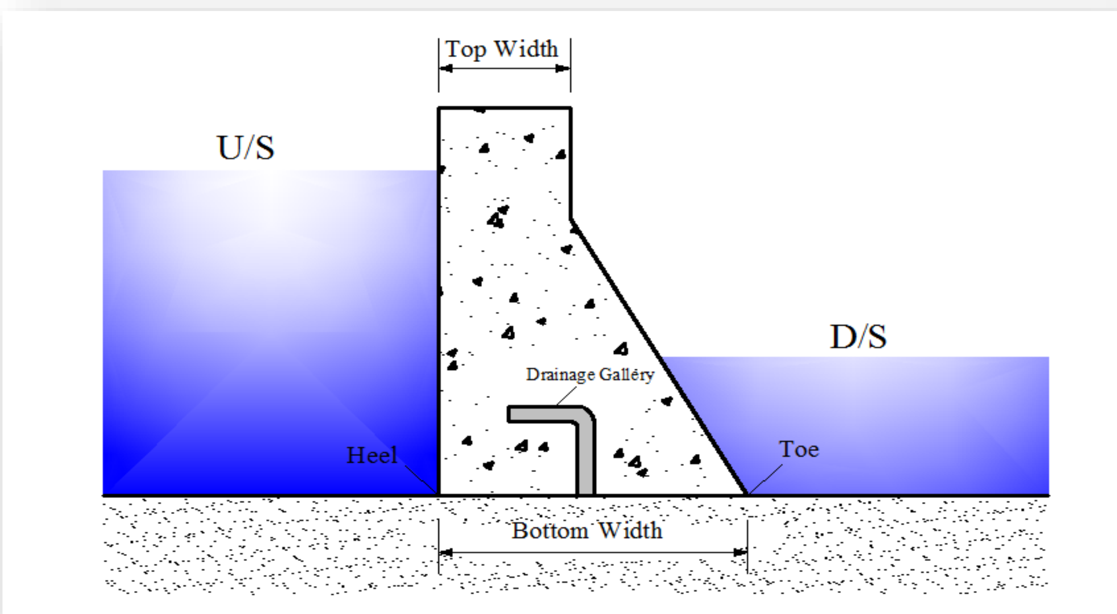


Figure (9.2): Profile in a Concrete Dam

9.7. Forces Acting on Gravity Dams

1. Water pressure
2. Uplift pressure
3. Silt pressure
4. Wave pressure
5. Pressure due to earth quake force
6. Ice pressure
7. Weight of the dam

1. Water Pressure

$P = 1/2\ \gamma\ h^2$

If the upstream face is partly vertical and partly inclined, the resultant water pressure can be solved in tow components

$\rightarrow P = \frac{1}{2}\ \gamma\ h^2$ & $P_1 = \gamma v \downarrow$

$\leftarrow P_2 = \frac{1}{2}\ \gamma\ h_1^2$ & $P_3 = \gamma v_1 \downarrow$

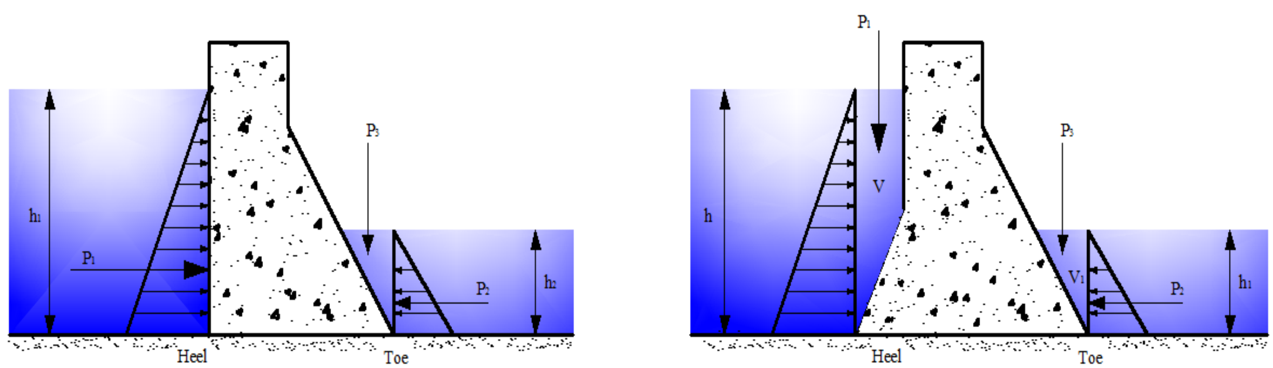


Figure (9.3): Water Pressure Distribution

2. Uplift Pressure

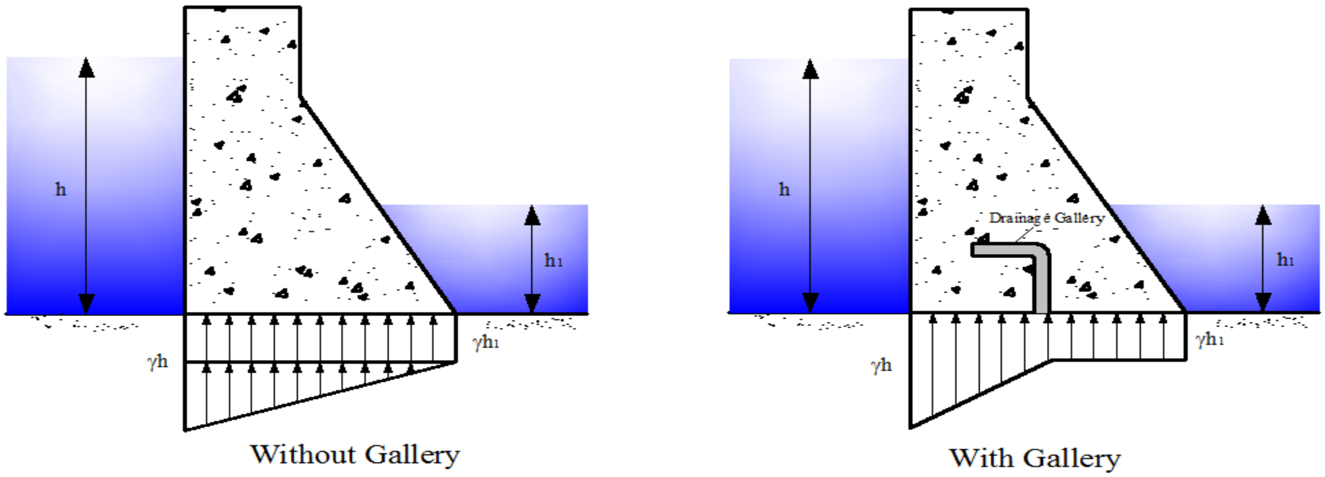


Figure (9.4): Uplift Pressure Distribution

$$\gamma h_1 + \frac{1}{3}(\gamma h - \gamma h_1) = \gamma(\frac{(2h_1 + h)}{3})$$

3. Earthquake Forces

a. Effect of vertical acceleration:

When the acceleration is vertically upward the inertia force:

$F_V = W\ K_v$

W: Weight of the dam

K_v: Coefficient of earthquake acts vertically downwards, these increasing the downwards weights.

When the acceleration is vertically downward the inertia force acts upwards and decrease the downward weight.

Net Weight = $W\ (1 \pm K_v)$

Note: K_v is (+) for acceleration is vertical upward and (-) for acceleration is vertical downward.

b. Effect of Horizontal Acceleration

I. Hydrodynamic Pressure

The horizontal acceleration of the dam and foundation towards the reservoir causes a momentary increase in the water pressure. The increase in water pressure (P_e) is given by:

$$\mathbf{P_e = 0.55\, K_h\, \gamma\, h^2} \qquad \left(\text{acts at } \frac{4h}{3\pi}\right)$$

K_h = coefficient of earthquake at horizontal direction

II. Horizontal Inertia Force

The inertia force acts in a direction opposite to the acceleration imparted by the earth quake forces

$$\mathbf{F_H = W\, K_h}$$

W = weight of the dam (This force can be considered at the center of gravity of the mass)

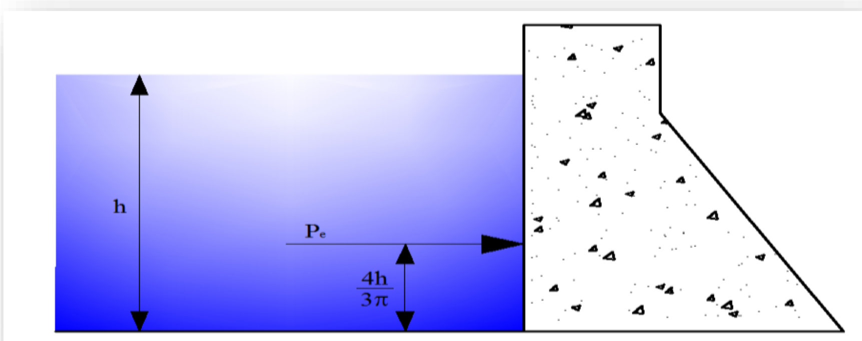


Figure (9.5): Location of Hydrodynamic Resultant

4. Wave Pressure

Wave pressure depends on the height of the wave (h_w) developed

$$\mathbf{h_w = 0.032\sqrt{(V * F)} + 0.763 - 0.271\sqrt[4]{F}} \qquad (\text{for } F \le 32 \text{ km})$$

$$\mathbf{h_w = 0.032\sqrt{(V * F)}} \qquad (\text{for } F > 32 \text{ km})$$

h_w = height of the wave in (meter)

V = wind velocity in (km / hr)

F = straight length of water expanse in (km)

∴ Wave pressure (P_W) = 2000 γ h_w^2 kg / m = 2 γ h_w^2 Ton / m

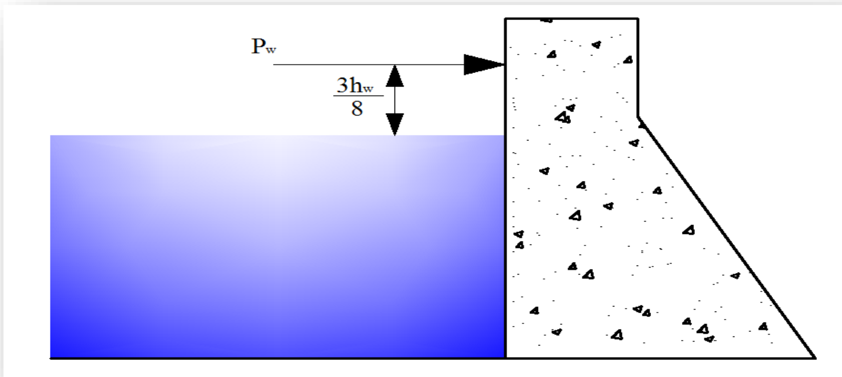


Figure (9.6): Location of Wave Pressure Resultant

5. Silt Pressure

$$P_{silt} = \frac{1}{2} K_a \gamma_s h_s^2$$

$$K_a = \frac{1-\sin\emptyset}{1+\sin\emptyset}$$

γ_s: Submerged unit weight of silt material

h_s: Height of silt

If the upstream face of the dam is inclined, the vertical weight of silt supported on the slope also acts as vertical force

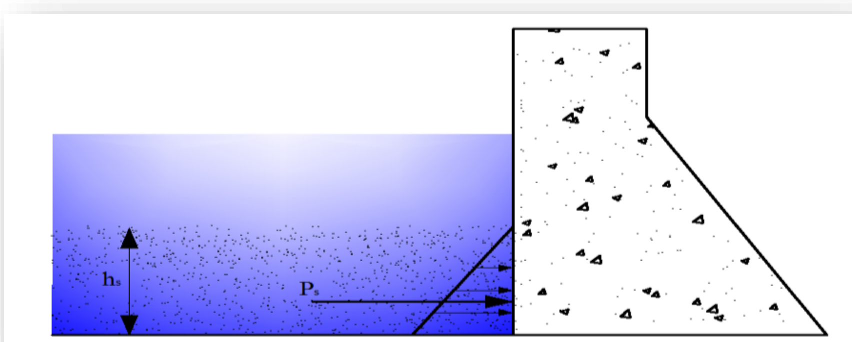


Figure (9.7): Location of Silt Pressure Resultant

6. Ice Force

The coefficient of the thermal expansion of ice being five times more than that of concrete. The ice force acts linearly along the length of the dam at the reservoir level. The average value of (5 Kg / cm^2) or (50 Ton / m^2) may be taken as an ice force.

7. Weight of the Dam

$W_1 = V_1\, \gamma_{con}$; $W_2 = V_2\, \gamma_{con}$

$W_{dam} = \sum W = W_1 + W_2 + W_3 + \ldots..$

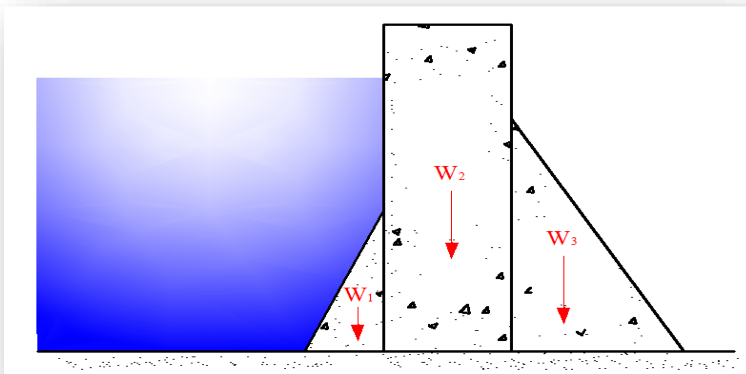


Figure (9.8): Weight of Gravity Dam

9.8. Structural Stability of Gravity Dams

Failure and inertia for structural stability of concrete gravity dam due to the following reasons:

1. Overturning of the dam
2. Compression or crushing of the dam
3. Sliding of the dam
4. Development of tension in the dam

1. Failure by Overturning

If the resultant of all forces acting on a dam at any section of its sections passes outside the toe and the dam shall rotates and overturning about the toe. The factor of safety against overturning is:

$(F.S)_{overturning}$ = (∑ Rotating moments) / (∑ Overturning moments)

$(F.S)_{overturning} = (\sum M_R) / (\sum M_O)$

The value of F.S against overturning should not be less than (1.5)

2. Compression or Crushing

A dam may fail by the failure of its materials, the compressive stress produced may exceed the allowable stress and dam material may get crushed. The vertical stress distribution at the base is given by:

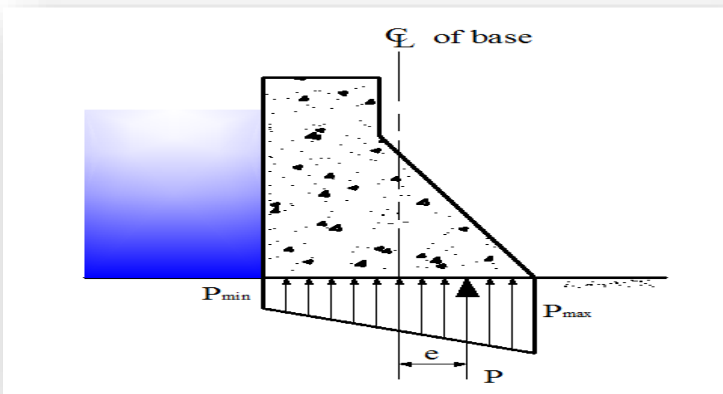


Figure (9.9): Vertical Stress Distribution at the Base

$$P_{max,\,min} = \frac{\sum V}{B} \mp \frac{M\,C}{I}$$

$\frac{\sum V}{B}$: Direct Stress

$\mp \frac{M\,C}{I}$: Bending Stress $= \mp \frac{V\,e}{\frac{1}{6} B^2} = \mp \frac{6\,V\,e}{B^2}$

+ will be used for calculating normal stress at the toe

- will be used for calculating normal stress at the heel

e: eccentricity of the resultant from the center of the base

$\sum$ V: total vertical force

B: base width of the dam

3. Sliding (Shear Failure)

Sliding occur when the net horizontal force at the base of the dam exceeds the frictional resistance developed at the level

$$\textbf{F.S}_{\text{sliding}} = \frac{\mu \sum(V-U)}{\sum H} > 1.0$$

$\sum$ (V-U): net vertical force = $\sum$ V

$\sum$ H: sum of horizontal forces causes the sliding

μ: coefficient of friction = (0.65 – 0.75)

4. Tension

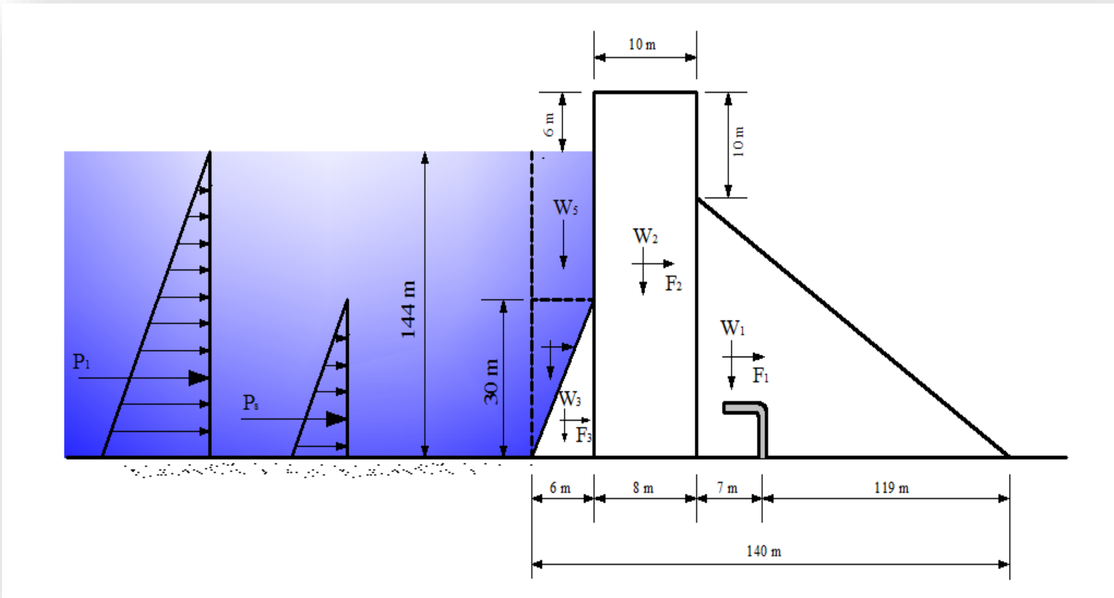
$$\textbf{P}_{\text{heel}} = \left(\frac{\sum V}{B}\right) * \left(1 - \frac{6e}{B}\right)$$

- If e > ($\frac{B}{6}$), the normal stress at the heel will be (-ve) or tension and no tension should be permitted at any point of the dam.
- The eccentricity (e) should be less than $\frac{B}{6}$ (i.e. (e < B/6)) → the resultant should always lie within the middle third.

Example 9.1: **Determine the heel and toe stresses and the factor of safeties for sliding and overturning for the gravity dam section shown in the figure below for the following loading conditions:**

- Horizontal earthquake (K_h) = 0.1
- Normal uplift pressure with gallery drain working
- Silt deposit up to 30 m height
- No wave pressure and no ice pressure
- Unit weight of concrete = 2.4 Ton/m^3 and unit weight of silty water = 1.4 Ton/m^3
- Submerged weight of silt = 0.9 Ton/m^3

- Coefficient of friction = 0.65 and angle of repose = 25°



Solution:

Name of forces	Magnitude	L_a (m)	Moment toe
1- vertical forces			
W_1	(126+140) * 0.5 * 1*2.4 = 21168 +↓	84.3	+1778112
W_2	150 * 8 *1*2.4 = 2889+↓	130	374400
W_3	(30+6)*0.5*1*2.4 = 216 +↓	136	29376
W_4	(30+6)*0.5*1*1.4 = 126 +↓	138	17388
W_5	114 *6*1*1 = 684 +↓	137	93708
$\sum W$	+ 25074		$\sum M$ = 229284
2- Uplift Pressure			
U_1	⅓ (1)(144)(21)(1) = - 1008 ↑	129.5	- 130536
U_2	½ (⅓ *1*144)*119*1 = - 2856 ↑	79.33	- 226576

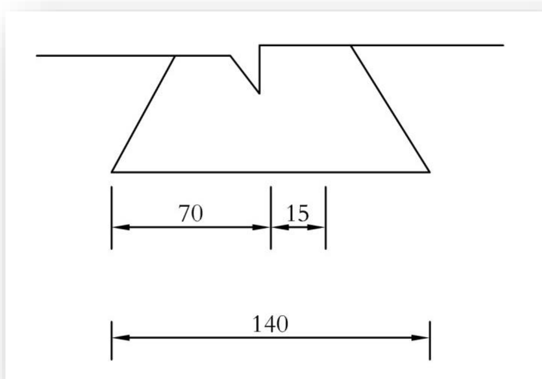
U_3	½ (⅔*1*144)*21*1 = -1008 ↑	133	- 134065
$\sum U$	- 4872		$\sum M$ = -491176
3- horizontal forces $P_1 = \frac{1}{2}\gamma h^2$ $P_s = \frac{1}{2}\gamma_s h_s^2 k_a$	 0.5 * 144^2 * 1*1 = - 10368 0.5 *0.9 * 30^2 *0.4058 = - 164.4	 144/3 30/3	 - 497664 - 164.4
$\sum H$	- 10532.4		- 499014
4- earth quake $F_1 = W_1 * K_h$ $F_2 = W_2 * K_h$ $F_3 = W_3 * K_h$ Hydrodynamic (P_e) ($0.55\, k_h\, \gamma\, h^2$)	 21168 *0.1= - 2116.8 2880*0.1 = - 288 216 *0.1 = - 21.6 0.55 * 0.1 *1 * 144^2 = - 1150.848	 140/3 150/2 30/3 144/3π	 - 98784 - 21600 - 216 - 703346
$\sum E$	- 3577.284		$\sum M$ = - 190934.6

$\sum M = +2292984 - 499014 - 190934.6 - 491176 = 1111859.4$ Ton. m

$\sum V = \sum W - \sum U = 25074 - 4872 = 20202$ Ton

$e = (B/2) - X' = (B/2) - (\sum M / \sum V)$

$e = (140/2) - (1111859.4 / 20202) = 15$ m



$P_{max,\ min} = (\sum V / B)(1 \pm ((6e)/B))$

$= (20202/140)(1 \pm ((6*15)/140))$

$P_{max} = 237.06$ Ton / m2

$P_{min} = 51.54$ Ton / m2

$F.S_{sliding} = (\mu \sum (V-U)) / (\sum H) \quad > 1.0$

$= 0.933 < 1$ not o.k

$(F.S)_{overturning} = (\sum M_R) / (\sum M_O) = 2292984 / (Mu + M_H + M_E)$

Chapter Ten

Design of Spillway

10.1. Introduction

A spillway is a structure constructed at a dam site through which the design flood could be disposed of safely to the downstream, just after the reservoir gets filled up. Up to the normal pool level, water starts flowing over the top of the spillway crest. A spillway is essentially a safety valve from a dam.

Figure (10.1): Stepped Spillway

In general, spillways comprise five distinct components namely:

1. Entrance channel
2. Control structure
3. Discharge carrier
4. Energy dissipator
5. Outlet channel

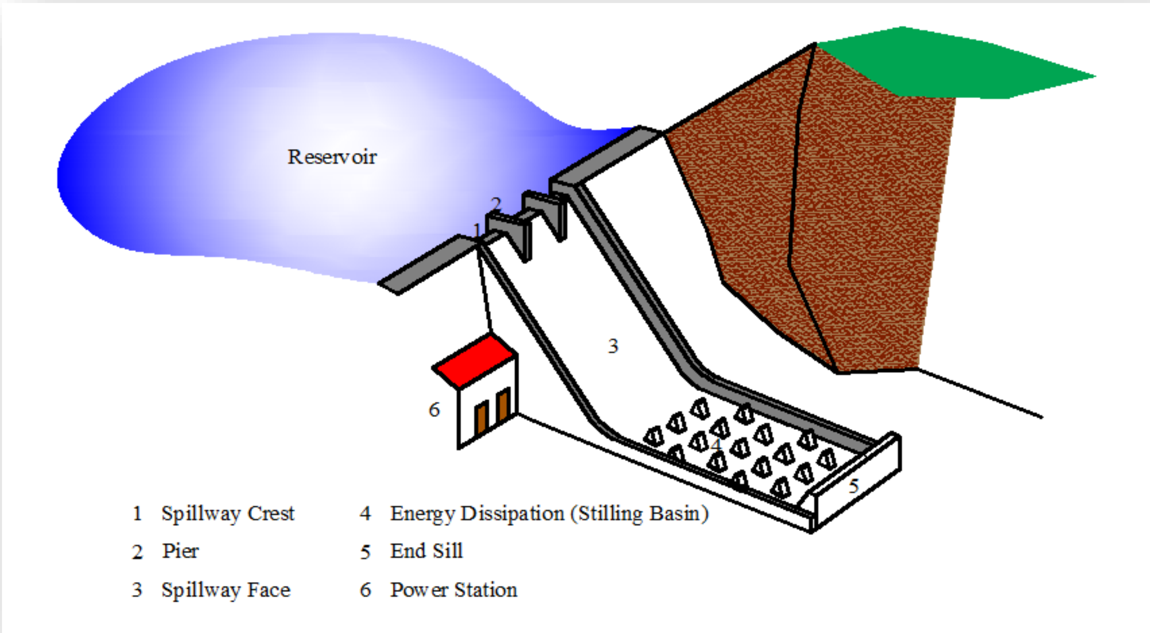


Figure (10.2): Component of Spillway

The entrance channel transfers water from the reservoir to the control structure, which regulates the discharge from the reservoir. Water is then conveyed from reservoir to the low-level energy dissipater on the riverbed by the discharge conveyor. An energy dissipater is required to reduce the high velocity of the flow to a non-scouring magnitude.

The crest of the spillway is usually provided at F.R.L (Full Reservoir Level) or (Normal Pool Level). However, in order to control floods the gates could be provided at the top and the water level could be increased up to maximum water level. The height between F.R.L and M.W.L is called the "Flood lift". Reservoir level should not cross MWL. Following are different types of spillways usually adopted in practice.

A spillway can be located either within the body of the dam or at one end of it or entirely away from it.

10.2. Classification of Spillways

The spillways can be classified of the following major types, depending upon the type of the structure constructed for disposing of the surplus water:

1. Straight drop spillway
2. Ogee spillway (overflow spillway)
3. Chute spillway (open channel spillway)
4. Side channel spillway
5. Shaft spillway
6. Siphon spillway

Major dam will be usually provided with an overflow spillway with crest gates. However, the type and location of spillway depends on the site conditions of topography.

10.3. Ogee Spillway (Overflow Spillway)

Ogee spillway is an improvement upon the free over fall spillway, and is widely used with concrete, arch and buttress dams.

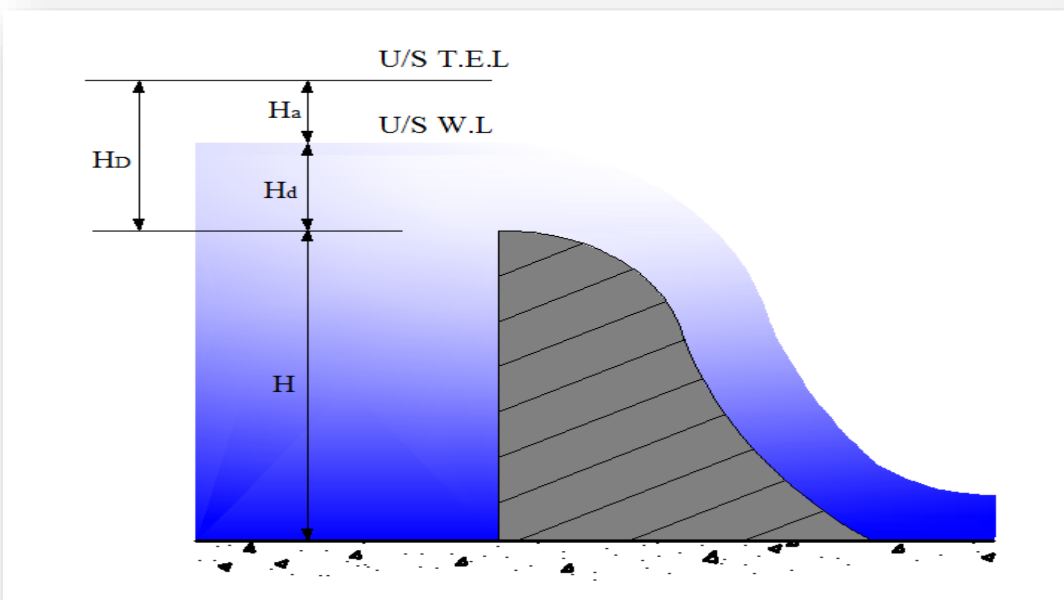


Figure (10.3): Ogee Spillway

The discharge passing over the ogee spillway is given by the equation weir:

$$Q = C\, L_e\, H_D^{3/2}$$

Where:

Q: Discharge.

L_e: Effective length of the spillway crest.

C: Coefficient of discharge

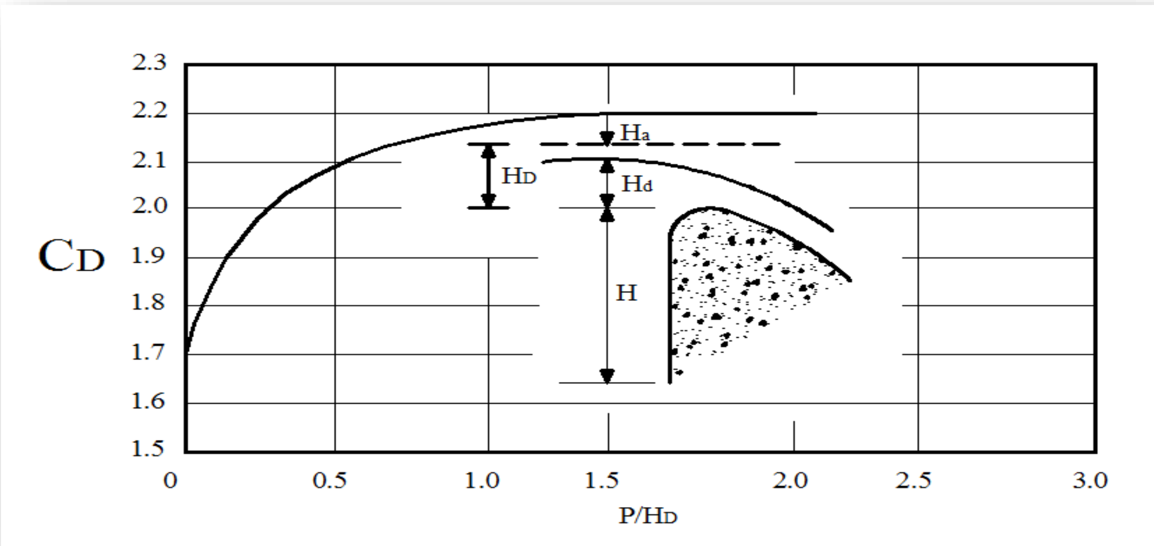


Figure (10.4): The Relation between C_D and P/H_D

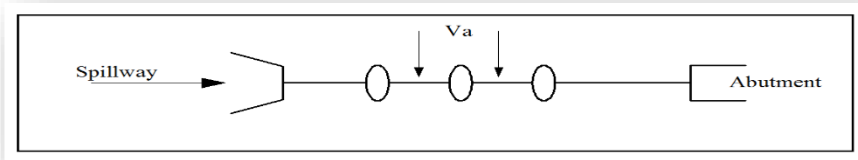
H_D: Total head over the crest = $(H_a + H_d)$

$$H_a = \frac{V_a^2}{2g}$$

$$V_a = \frac{Q}{(H + H_d)B}$$

B: width of canal

$$L_e = L - 2\,[N{*}K_p + K_a]\,H_D$$



L: The net clear length of the spillway crest

K_p: Pier contraction coefficient

N: Number of piers

K_a: Abutment contraction coefficient

Table (10.1): Values of K_p and K_a According to the Shape of Pier and Abutment

Pier Shape	**K_p**
Square nosed piers without any rounding	0.10
Square nosed piers with rounding on radius at corners =0.1 of pier thickness	0.02
Rounded nose piers and 90° cut water nosed piers	0.01
Pointed nose piers	0.00
Abutment Shape	**K_a**
Square abutment with head wall at 90° to the direction of flow	0.2
Rounded abutment with head wall at 90° to the direction of flow	0.1

For high spillway the approach velocity is very small and the velocity head can be neglected.

$\therefore (H_a = 0) \longrightarrow (H_D = H_d)$

If $H / H_d > 1.33$ (High ogee spillway)

10.3.1. Crest of Spillway

For spillway having a vertical face the D/S crest is given by:

$$X^{1.85} = 2\,H_d^{0.85}\,y$$

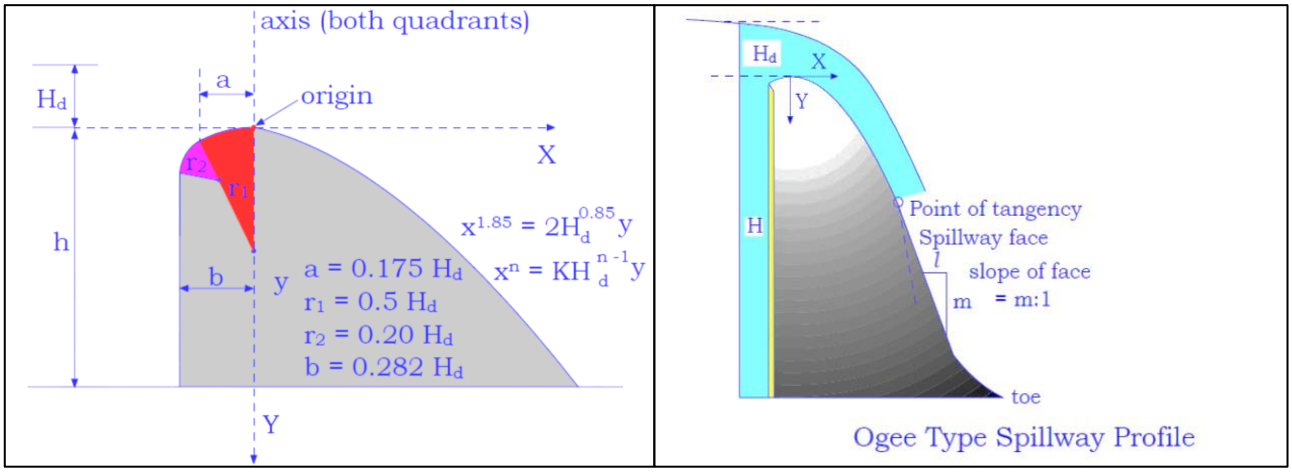


Figure (10.5): Details of Crest of Spillway

The U/S profile may be designed as the equation:

$$y = \frac{0.724(x+0.27H_d)^{1.85}}{H_d^{0.85}} + 0.126H_d - 0.4315H_d^{0.375}(x+0.27H_d)^{0.625}$$

Example 10.1: **Design ogee spillway for concrete gravity dam having downstream face sloping at (0.781H: 1V), the discharge for the spillway is (8000 m^3/sec) the height of spillway crest is kept at level (204 m) the average river level at the site is (100 m), the spillway length consist (6) span having clear width (10 m) each with pier thickness=3m, k_p=0.01, k_a=0.1, C=2.2.**

Solution:

N=5

L = 6 * 10 = 60 m

$L_e = L – 2 [N*K_p + K_a] H_d = 60 – 2 [5 (0.01) + 0.1] H_d = 60 – 0.3 H_d$

$Q = C\ Le\ H_D^{3/2}$

$8000 = 2.2 (60 – 0.3 H_e) (H_e^{3/2})$

By trial and error

$H_D = 16.3$ m

Let $H_D = H_d$

$$V_a = \frac{Q}{(H + H_d)B}$$

$$V_a = \frac{8000}{((204-100) + 16.3)(60+5*3)} = 0.887 \text{ m/sec}$$

$H/H_d = (204 – 100)/16.3 = 6.4 > 1.33$ » High spillway

$$H_a = \frac{V_a^2}{2g} = \frac{0.887^2}{2*9.81} = 0.04 \text{ m}$$

∴ This is very small value and may neglected

D/S Profile

$\mathbf{X^{1.85} = 2\ H_d^{0.85}\ y}$

$y = X^{1.85} / 2\ H_d^{0.85}$

$y = X^{1.85} / 2\ (16.3)^{0.85}$

$y = X^{1.85} / 21.45$

$dy/dx = (1.85\ X^{0.85}) / 21.45$

$1/0.781 = 0.0862\ X^{0.85} \rightarrow X = 22.4$ m

The coordinates of the D/S profile are:

X	0	2	4	6	8	10	12	14	16	18	20	22	22.4
$y = \frac{x^{1.85}}{21.45}$	0	0.17	0.61	1.28	2.18	3.3	4.62	6.15	7.87	9.79	11.9	14.19	14.67

U/S Profile

$$y = \frac{0.724(x+0.27H_d)^{1.85}}{H_d^{0.85}} + 0.126H_d - 0.4315H_d^{0.375}(x + 0.27H_d)^{0.625}$$

$$y = \frac{0.724 (x + 0.27 *16.3)^{1.85}}{16.3^{0.85}} +0.126*16.3 - 0.4315 *16.3^{0.375} (x + 0.27 *16.3)^{0.625}$$

The values U/S profile extends up to

$X = - 0.28*H_d = - 0.28*16.3 = - 4.56$ m

X	0	-0.5	-1	-1.5	-2.0	-2.5	-3.0	-3.5	-4.0	-4.56
Y	0	0.01	0.06	0.14	0.27	0.43	0.66	0.95	1.37	2.06

H.W: Design ogee spillway for a concrete gravity dam and find the maximum flood discharge could be disposed of safely to the downstream. It is having downstream face sloping at (0.85H: 1V), the height of spillway crest is kept at (0.85H:1V), the height of spillway crest is kept at level (310 m), the average river bed level at the site is (190 m), the spillway length consist (5) span having clear width (12 m) each with pier thickness = 3m , k_p=0.01 , k_a=0.1 , C=2.2 . If the D/S crest profile is given by: $x^{1.85} = 25$ y , and the U/S by:

$$y = \frac{0.724\,(x+0.27\,H_d)^{1.85}}{H_d^{0.85}} + 0.126\,H_d - 0.4315\,H_d^{0.375}\,(x + 0.27\,H_d)^{0.625} \quad .$$

Neglect the approach velocity head